ROCK AND RIPRAP DESIGN MANUAL
FOR CHANNEL EROSION PROTECTION

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by

William C. Hughes
University of Colorado

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LIST OF SYMBOLS

A = Channel cross sectional Area - ft^2

A' = Ripraped apron thickness (scoured basins) - ft

a = Water area of partially filled circular conduit - ft^2

B = Ripraped apron end slope thickness - ft

D = Circular conduit diameter - ft

D_x = Filter particle size, where x is the percent by weight finer than D - mm

d_s = Depth of channel scour - ft

d_t = Tailwater depth - ft

d_x = Soil particle size, where x is the percent by weight finer than d - mm

E = Vertical drop of ripraped basin end slope - ft

EOS = Filter cloth equivalent opening size

F = Force - lb

F_B = Riprap coefficient for long channel bends

F'_B = Riprap coefficient for short channel bends

F_D = Drag force - lb

F_I = Dimension multiplier

F_o = Conduit outlet Froude number

H_o = Vertical dimension of a rectangular conduit - ft

K = Rock or riprap particle size - inches

K_E = Roughness element height - inches

K_m = Effective rock or riprap size - inches

L = Riprap apron length - ft

L_s = Length of scour hole - ft

n = Channel roughness coefficient

Q = Channel discharge - cfs

R = Channel hydraulic radius - ft

R_d = Radius of channel bend

S = Slope of energy grade line or friction slope

S_o = Bottom slope of channel

S_s = Rock specific gravity

S.F.= Riprap safety factor

T = Riprap blanket thickness - ft

T_W = Channel water surface width - ft

V = Channel mean velocity - ft/sec

W_b = Width of ripraped apron - ft

W_o = Width of a rectangular conduit - ft

W_s = Width of scour hole - ft

Y = Vertical depth of flow - ft

Y_c = Critical depth - ft

Y_n = Normal depth of flow - ft

Y_o = Depth of flow at a conduit outlet - ft

Y_1 = Depth of flow over roughness element - ft

γ = Unit weight of water

Δ = Internal angle of a channel bend (see figure IV-6)

Δ_c = Centerline angle of a channel bend (see figure IV-6)

η = Channel stability number

η' = Side slope stability number

θ = Ripraped basin flare angle

LIST OF SYMBOLS (Continued)

θ' = Channel side slope angle

λ = Downslope angle of the channel velocity vector

ρ = Density

τ_s = Average tractive force 1/ft

Φ = Riprap angle of repose

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I - INTRODUCTION

Urbanization and roadway development inevitably produce greater volumes of flow and higher peak discharges in adjacent streams. An unprotected channel is gradually enlarged because of the increased flow, particularly increased flood flows, and tends to have unstable and unvegetated banks, scoured or muddy channel beds, an accumulation of debris and a high sediment concentration. Probably the best method to alleviate these problems is to control the increased flows before they reach the stream channel, possibly with detention ponds. However this technique is often not feasible. In addition to increasing stream flows, urbanization and roadways produce numerous locations where flows are concentrated sufficiently to produce erosive velocities in the stream channel. Examples are roadway cross drains, drainage outfalls and detention pond outlets. Where it is not possible to use grass or other natural cover to protect the channel, rock and wire bound rock, provide a relatively inexpensive and aesthetically pleasing lining material, and when placed in the proper configurations can effectively dissipate much of the energy inherent in high velocity flows to produce tranquil, less erosive flow conditions.

Although the use of rock for erosion protection in channels precedes written history, very little design

information has evolved, and much of what has been developed is scattered through numerous journals and reports, and is not readily available to the design engineer. The purpose of this manual is to compile the available design information for either riprap or wire bound rock channel linings and energy dissipators into a usable design format. This has been done by representing much of the design information in graphical form. Condensing information into a graphical form necessitates a certain amount of smoothing of the data which introduces some degree of error. In most cases the inaccuracies resulting from smoothing are no greater than those which are inherent in simply reading a graph and are generally much smaller than the inaccuracies associated with determining the design discharge for a stream channel. Where possible the original information from which the graphs were developed is included in the appendix of this manual.

This manual is intended to supplement standard hydraulics texts and manuals and assumes the user is acquainted with the methods of open channel hydraulics and hydraulic design. Each chapter covers a different aspect of erosion protection using rock and wire bound rock, with a brief description of the design method at the beginning of the chapter, followed by several example problems. The design charts and nomographs associated with each chapter are bound together at the end of the chapter.

II - CHANNEL EROSION

Estimating whether an anticipated channel modification will result in downstream channel erosion is probably less exact and more difficult than the actual design of the appropriate erosion control measure. To an unfortunately large degree estimating whether erosion will result, and more importantly its degree of severity, depends on judgment based on experience with similar soils and situations.

The purpose of this chapter is to discuss briefly two catagories of channel erosion often encountered and describe the various control measures appropriate for each. In addition, the chapter includes tables of permissible velocities and curves relating erosion potential to channel slope and discharge, which are essentially a compila of experience curves from a number of investigators and from a number of geographical locations. The permissible velocity tables include somewhat expanded Fortier and Scobey tables which can be considered quite general, and values determined by Keeley in Oklahoma which Keeley warns may contain a geographical bias (17). In any case, it should be recognized that these curves and tables are guides and cannot replace experience for a particular location and situation.

Classification of Channel Erosion

Channel erosion for engineering purposes can be categorized as one of two scour conditions - Gully Scour and a Scour Hole.

Scour Hole - Localized scour in an otherwise stable channel resulting from high velocities such as encountered at culvert outlets. Scour holes may be several feet deep but generally extend down stream only a short distance. Scour holes are excellent energy dissipators but are unsightly and may cause structural damage by undercutting.

Scour holes result from the dissipation of high, localized kinetic energy and any erosion protection must deal with this problem, either by providing a protective lining of sufficient length that the kenetic energy has been reduced to permissible levels, or by dissipating the kinetic energy with a hydraulic jump or impact device. A third possibility is to use the scour hole itself as an energy dissipator, with or without a protective lining. In the latter case a cutoff wall to about 0.7 the depth of the expected scour hole should be considered to prevent undercutting.

Gully Scour - Extensive channel scour indicating an unstable channel in which the gradient is too great for the given bed or bank materials and flow rates. Soil erosion displaces channel material to form a ravine or

gully which may be of considerable length beginning at a downstream stable control section and extending upstream to the structure. This condition is not one of excessive velocity as much as increased flow rates due to the concentration of runoff into a single channel.

Erosion control or protection is different for bully scour conditions than a scour hole, although it is possible to have both conditions simultaneously, i.e., an unstable channel due to concentration of runoff and excessive velocities at a structure outlet. In general, gully scour protection measures include either changing the bed and banks to a stable material, that is lining the channel, or changing the channel slope to one that is stable using drop structures or check dams. The stable design gradient can be based on permissible velocities if the bed slope is not too steep. The following has been found acceptable for determining the spacing of check dams in mountainous terrain (16):

$$S_{design} = 0.7 \times S_o \text{ where } S_o < 20\%$$

$$S_{design} = 0.5 \times S_o \text{ where } S_o \geq 20\%$$

where

S_{design} = design gradient for establishing check dam spacing

and

S_o = the original channel gradient.

EXAMPLE II-1

Given: A natural channel is approximately trapezoidal in shape, with a 5 ft base width and 3:1 side slopes. The channel bed and banks are a sandy-silt and the channel gradient is about 0.3 percent.

Find: Determine whether the channel is likely to be stable at a design discharge of 40 cfs.

Using Figure II-1, 40 cfs and a channel slope of 0.3 percent is stable. Check using Table II-1. Assuming uniform flow in the channel and using Manning's equation;

$$40 = \frac{1.49}{0.03} \left[(5+3Y)Y\right] \left[\frac{(5+3Y)Y}{5 + 2Y\sqrt{10}}\right]^{2/3} (.003)^{1/2}$$

yields a normal depth, $Y_n = 1.53$ ft.

From Table II-1 the maximum permissible velocity is: $V = cd^{0.2}$ or

$$V_{max} = 2.5(1.53)^{0.2} = 2.72 \text{ fps.}$$

Channel velocity is: $V = \frac{Q}{A}$ or

$$V_{channel} = \frac{40}{(5+3Y)Y} = 2.73 \text{ fps.}$$

Comparison indicated channel is marginally stable at 40 cfs.

PERMISSIBLE VELOCITY

CHANNEL MATERIAL	MAXIMUM PERMISSIBLE VELOCITY fps[a]	MAXIMUM PERMISSIBLE VELOCITY fps[b]
Fine Sand	2	
Coarse Sand	4	
Fine Gravel < 3/4"	6	
Sandy Silt	2	
Sand Silt-PI < 6		2.5
Silt Clay	3.5	
Silt Clay-PI 6-10		2.5
Clay	6	
Clay-PI > 10		3.5
Poor Rock (usually sedimentary)	10	
Soft Shale	3.5	3.2
Soft Sandstone	8	3.8
Good Rock	20	
Bermuda Grass	(5 poor cover)	
5% Slope		
Sandy Silt	6 (5 poor cover)	
Silt Clay	6 (5 poor cover)	
Kentucky Bluegrass		
5% Slope		
Sandy Silt	5	
Silt Clay	7 (5 poor cover)	

[a]After Fortier and Scobey (54). Values given are for straight channels and depths less than 3 ft.

[b]After Keeley (17). Values given are for a depth of flow of one foot and are equal to C in Keeley's permissible velocity formula: $V = Cd^{0.2}$ where d = depth of flow.

TABLE II-1

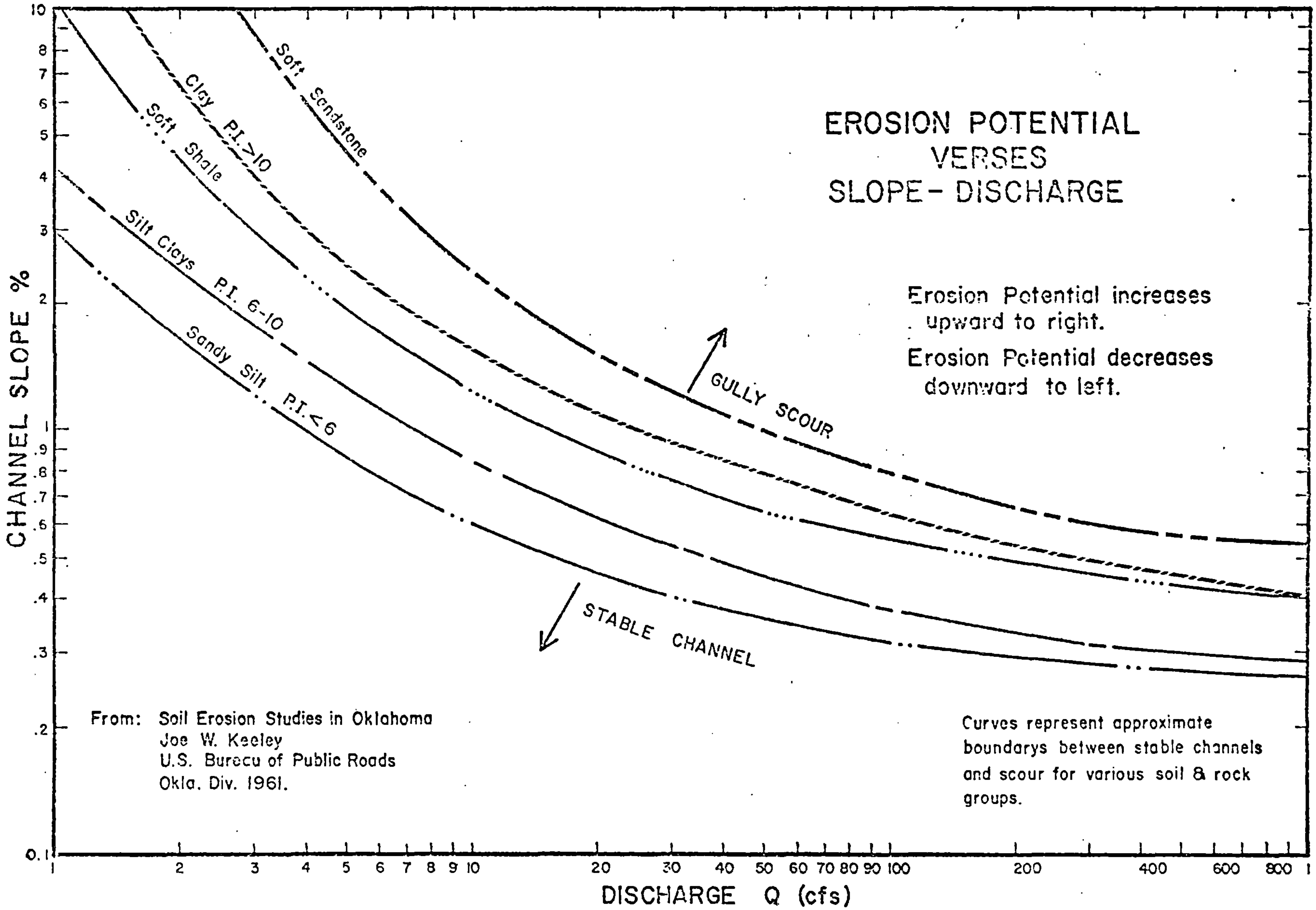

EROSION POTENTIAL
VERSES
SLOPE- DISCHARGE
Erosion Potential increases upward to right.
Erosion Potential decreases downward to left.
GULLY SCOUR
STABLE CHANNEL
Soft Sandstone
Clay P.I.>10
Soft Shale
Silt Clays P.I. 6-10
Sandy Silt P.I.<6
CHANNEL SLOPE %
DISCHARGE Q (cfs)
10
8
7
6
5
4
3
2
1
.9
.8
.7
.6
.5
.4
.3
.2
0.1
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
200
400
600
800
1
From: Soil Erosion Studies in Oklahoma
Joe W. Keeley
U.S. Bureau of Public Roads
Okla. Div. 1961.
Curves represent approximate boundarys between stable channels and scour for various soil & rock groups.

Fig. 11-1

III - FILTERS

An entire chapter has been devoted to the design of filters since the long term stability of riprap, or wire encased rock, when used as a protective lining, energy dissipator apron, or other structure, depends on a sound foundation - in this case a proper filter.

In a stream of flowing water, pressures on the boundary are not exactly hydrostatic. At some points the pressure will be greater than hydrostatic and at others less. The location of these points of pressure deviations may be relatively fixed with respect to an object, or transitory due to turbulence. This relative difference in boundary pressure produces an underflow through a previous lining, such as riprap or wire bound rock, and results in upward velocities of sufficient strength to remove bed material (30). A properly designed filter prevents fine bed material from washing up through the riprap pores but allows water to pass upward without creating damaging uplift pressures.

Two types of filters are now in common use: 1) Granular reverse filters originally developed by Terzaghi and later modified by the Corps of Engineers Laboratory at Vicksburg, Miss., sometimes referred to as the Terzahgi-Vicksburg (T-V) filter, and 2) Plastic Filter Cloth which is a more recent development now available in several mesh sizes.

Filters are not always required. The grading of the bed material may be such that it is very nearly a natural filter, or in some cases a highly cohesive soil is sufficiently stable that a filter is unnecessary. Except for highly cohesive soils the criteria for the Terzaghi-Vicksburg reverse filter provides a good test as to whether a filter is required.

Granular Reverse (T-V) Filters

The granular reverse filter consists of one or more layers of graded granular material, layered in such a way that it constitutes an inverted filter which will not allow fine materials to wash through but allows water to pass upward without creating a pressure gradient that would jeopardize the stability of the bed. Layer thickness is generally no less than 6 inches because of restrictions in construction techniques.

The specifications for the T-V reverse filter relate the grading of the protective layer (filter) to that of the bed material (base) by the following inequalities:

$$D_{15} \leq 5\ d_{85} \qquad \text{(III-1)}$$

$$4d_{15} \leq D_{15} \leq 20d_{15} \qquad \text{(III-2)}$$

$$D_{50} \leq 25\ d_{50} \qquad \text{(III-3)}$$

where the capital "D" refers to the filter grain size and the lower case "d" to the base grain size. The

subscripts refer to the percent by weight which is finer than the grain size denoted by either "D" or "d". For example, fifteen percent of the filter material by weight is finer than D_{15} and eighty-five percent of the base material by weight is finer than d_{85}. Note that in multiple layered filters the base is the next lower filter layer for all but the bottom filter layer. It has been found that a filter which will protect a granular base with d_{50} of 0.045 mm will protect anything finer (27). Using this as a lower limit on grain size needing protection may reduce the number of filter layers required to protect a very fine base material.

As will be seen in the following examples, there is often considerable latitude allowed in the grading of the filter layers as specified by equations one, two and three. For this reason pit run (unwashed) material such as concrete sand makes an excellent filter.

Filter Cloth

Filter cloth is woven from synthetic fibers to produce a non-degrading open mesh filter and is generally sold in 100 foot long rolls, 12 to 18 inches in width. (A list of leading filter cloth manufacturers is given on page 65 of the May 1976 Civil Engineering-ASCE magazine.) When placed, the edges of adjacent sheets are overlapped about 8 inches and hand sewn in the field with nylon twine while the sheets held in place by 3/16

inch steel securing pins driven at approximately 3 ft centers. Filter cloth is more easily placed than granular filters and is less expensive if the material required for a granular filter must be processed or hauled very far. However, although filter cloth has been used quite successfully, it is <u>not</u> equivalent to a granular filter. Filter cloth is essentially a one-dimensional filter (perpendicular to the cloth), whereas a granular filter with its vertical thickness also acts as a filter for seepage flows parallel to the channel bottom. For this reason piping failures may result if seepage flows have significant components parallel to the filter cloth. In bed soils containing fine silts or clay, it has been found that the silt builds up or cakes along the lower surface of filter cloth reducing its permeability substantially and resulting in excessive hydrostatic pressures.

Filter cloth is rated according to its strength and mesh size; minimum strength specifications require that the tensil strengths in strongest and weakest directions be not less than 350 lb. and 200 lb., respectively. Elongation should not exceed 35 percent and the minimum bursting strength should be at least 520 psi. For design purposes mesh size needs to be given in terms of the percent of open area and the equivalent opening size (EOS), which is the U.S. standard sieve size which retains a sand of which 95 percent by weight is also retained on the filter cloth.

Design criteria for filter cloths are as follows (51):

A.) Filter cloth adjacent to granular materials containing 50 percent or less by weight fines (minus No. 200 sieve)

- EOS $\leq d_{15}$
- The area of openings should not exceed 67 percent of the total area.

B.) Filter cloth adjacent to other soils

- EOS $\leq$ No. 70 U.S. Standard Sieve
- The area of openings should not exceed 10 percent of the total area.

C.) In no case should the area of openings be less than 4 percent of the total area, nor should the EOS be less than the No. 100 U.S. Standard Sieve.

D.) In general, it is best to use filter cloth with maximum allowable opening size.

EXAMPLE III-1

Find the gradation band within which a granular reverse filter must lie to protect a channel with the sandy-silt gradation shown in Figure III-1.

From the gradation curve read:

d_{15} = .016 mm

d_{50} = .051 mm

d_{85} = 0.10 mm

Requirement 1. $d_{85} \times 5$ = upper limit of D_{15} or

$0.10 \text{ mm} \times 5 = 0.5 \text{ mm}$

Requirement 2. $d_{15} \times 20$ = upper limit D_{15}

$0.016 \times 20 = 0.32 \text{ mm}$

Since 0.32 mm < .5 mm requirement 2 controls and the upper limit for D_{15}

$\therefore D_{15} = 0.32 \text{ mm}$

Requirement 3. $d_{15} \times 4$ = lower limit D_{15}

$0.016 \times 4 = 0.064 \text{ mm}$

Requirement 4. $d_{50} \times 25$ = upper limit for D_{50}

$0.051 \times 25 = 1.28 \text{ mm}$

The required lower layer for the reverse filter must fall within the shaded band shown on Fig. III-1, and should have a gradation curve of similar shape to the bed material gradation curve.

Once a gradation curve for the material to be used as the lower layer of the reverse filter has been determined, the limits and resulting gradation band for the next layer of the reverse filter would be established following the same procedure. The number of layers required for the reverse filter depends on the size and/or gradation of the top layer of riprap, or wire bound rock. Additional layers will be required until the top layer (riprap) gradation curve falls within the gradation band established by the next lower layer.

Often the characteristics of the erodable channel materials are limited to a general soil classification

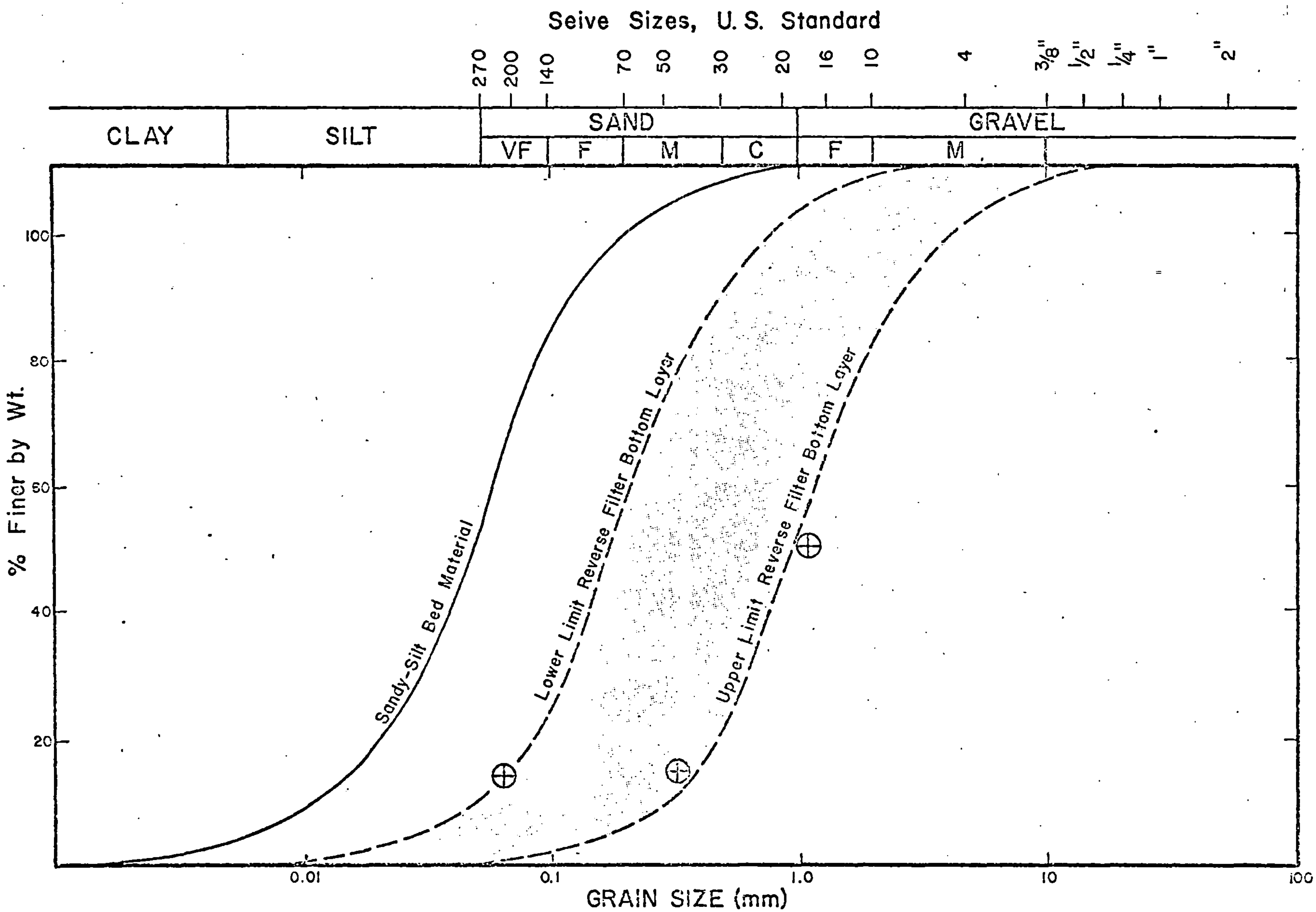

Seive Sizes, U. S. Standard
270
200
140
70
50
30
20
16
10
4
3/8"
1/2"
1/4"
1"
2"
CLAY
SILT
SAND
GRAVEL
VF
F
M
C
F
M
100
80
60
40
20
% Finer by Wt.
Sandy-Silt Bed Material
Lower Limit Reverse Filter Bottom Layer
Upper Limit Reverse Filter Bottom Layer
0.01
0.1
1.0
10
100
GRAIN SIZE (mm)

Fig. III-1

such as silty-sand, etc. In this case a well defined gradation as used in Example III-1 is not available and a second method for estimating the reverse filter gradation may be used. The second method begins with the riprap gradation and is essentially the reverse of the procedure used in Example III-1.

In the following example this procedure is used, and, in addition, the fact that a filter which protects a soil with d_{50} of 0.045 mm will protect anything finer is used to set a lower limit on the filter (27). Note, the symbol K is used to denote rock, or riprap, particle size in this manual.

EXAMPLE III-2

Find the required filter gradations to protect a bed soil with a mean diameter, d_{50}, at 0.045 mm or less, for a riprap cover with a maximum size of 6 in., and graded such that

$$K_{85} = 6 \text{ inches}$$
$$K_{50} = 4 \text{ inches}$$
$$K_{15} = 2 \text{ inches}$$

Requirement 1. $\dfrac{2 \text{ in. x } 25.4 \text{ mm/in}}{5} = D_{85}$ underlayer one (lower limit)

$= 10.2$ mm

Requirement 2. $\dfrac{2 \text{ in. x } 25.4 \text{ mm/in}}{4} = D_{15}$ underlayer one (upper limit)

$= 12.7$ mm

Requirement 3. $\frac{2 \text{ in. x } 25.4 \text{ mm/in}}{20} = D_{15}$ underlayer one (lower limit)

$= 2.5$ mm

Requirement 4. $\frac{4 \text{ in. x } 25.4 \text{ mm/in}}{25} = D_{50}$ underlayer one (upper limit)

$= 4.1$ mm

The above defines underlayer band number one. Assume gravel with gradation shown (underlayer one) in Fig. III-2 is available, where $D_{15} = 6$ mm and $D_{50} = 8$ mm. For the next lower layer

Requirement 1. $\frac{6 \text{ mm}}{5} = D_{85}$ underlayer two (lower limit)

$= 1.2$ mm

Requirement 2. $\frac{6 \text{ mm}}{4} = D_{15}$ underlayer two (upper limit)

$= 1.5$ mm

Requirement 3. $\frac{6 \text{ mm}}{20} = D_{15}$ underlayer two (lower limit)

$= 0.3$ mm

Requirement 4. $\frac{8}{25} = D_{50}$ underlayer two (upper limit)

$= .32$ mm

These computations define underlayer band number two. Assume a fine gravel - coarse-sand material is available for underlayer two, with gradation curve (underlayer two) as shown in Fig. III-2. Since $D_{50} =$ 1.3 mm in underlayer two and $\frac{1.3 \text{ mm}}{25} = .052$ mm which is greater than .045 mm, a third layer will be required.

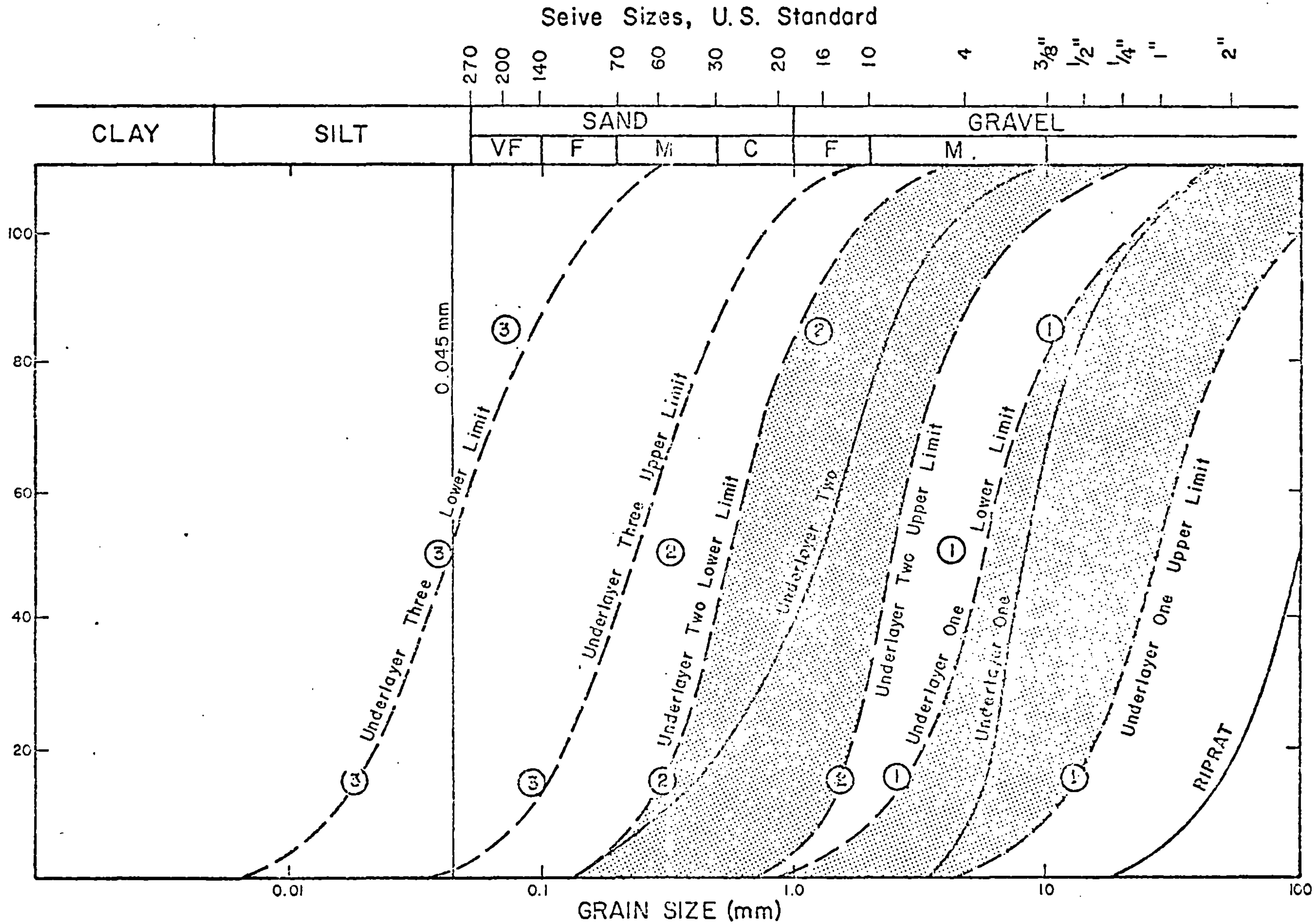
Requirement 1. $\frac{0.4 \text{ mm}}{5} = .08 \text{ mm} = D_{85}$ underlayer three (lower limit)

Requirement 2. $\frac{0.4 \text{ mm}}{4} = .10 \text{ mm} = D_{15}$ underlayer three (upper limit)

Requirement 3. $\frac{0.4 \text{ mm}}{20} = .02 \text{ mm} = D_{15}$ underlayer three (lower limit)

Requirement 4. $\frac{0.13 \text{ mm}}{25} = 0.52 \text{ mm} = D_{50}$ underlayer three (upper limit)

As before, these computations establish the band for underlayer three, which is shown in Figure III-2.



Seive Sizes, U.S. Standard
270
200
140
70
60
30
20
16
10
4
3/8"
1/2"
3/4"
1"
2"
CLAY
SILT
SAND
GRAVEL
VF
F
M
C
F
M.
0.045 mm
Underlayer Three Lower Limit
Underlayer Three Upper Limit
Underlayer Two Lower Limit
Underlayer Two
Underlayer Two Upper Limit
Underlayer One Lower Limit
Underlayer One
Underlayer One Upper Limit
RIPRAT
100
80
60
40
20
0.01
0.1
1.0
10
100
GRAIN SIZE (mm)

Fig. III-2

IV - CHANNEL LININGS

An effective method of minimizing (or entirely eliminating) erosion problems in a channel is to provide a protective lining of noneroding material on the banks and/or bottom of the Channel. A great variety of materials are available for this purpose, but in general, linings are categorized as either rigid or flexible.

Rigid Linings - as the name implies are stiff, unyielding linings usually constructed from materials such as portland cement concrete, masonry or asphaltic concrete. The hydraulic design of rigid channel linings is well documented with procedures discussed in most hydraulics texts. For this reason, design methods for rigid channel linings will not be included in this manual, however, rigid linings are mentioned because they have certain inherent advantages. They are durable, noneroding linings and are generally relatively smooth with a high flow capacity for a given cross-sectional area. In addition, rigid linings can be constructed with steep, even vertical, sidewalls, However, rigid linings also have disadvantages. They are expensive, have an unnatural appearance, prevent infiltration and may result in excessive erosion due to high velocities at the linings downstream end. Rigid linings are subject to failure due to undercutting and piping as well as uplift due to seepage beneath the lining.

Flexible Linings - again as the name implies are pliable linings which yield with changes in channel configuration. Riprap, wire encased rock and vegetation, usually grass, linings are relatively permanent flexible lining materials, while jute mesh, straw with erosion net, fiberglass mats and excelsior mats are considered temporary flexible linings which are normally used to protect a channel while vegetation or grass is being established. Flexible linings are not subject to undercutting or uplift failures being both pliable and porous, however they are subject to certain degree of scour. Generally, flexible linings must be constructed with gentle side slopes (3:1 or less), however wire encased rock can form a stable side wall with near vertical side slope.

Design procedures for vegetation, or grass linings, have been well documented and will not be duplicated here. A method for designing temporary linings is discussed in the above procedures, however permissible velocities over jute, excelsior mat and straw with erosion net have not been generally available until recently. Therefore a table of permissible velocities for those temporary flexible linings is included at the end of the chapter.

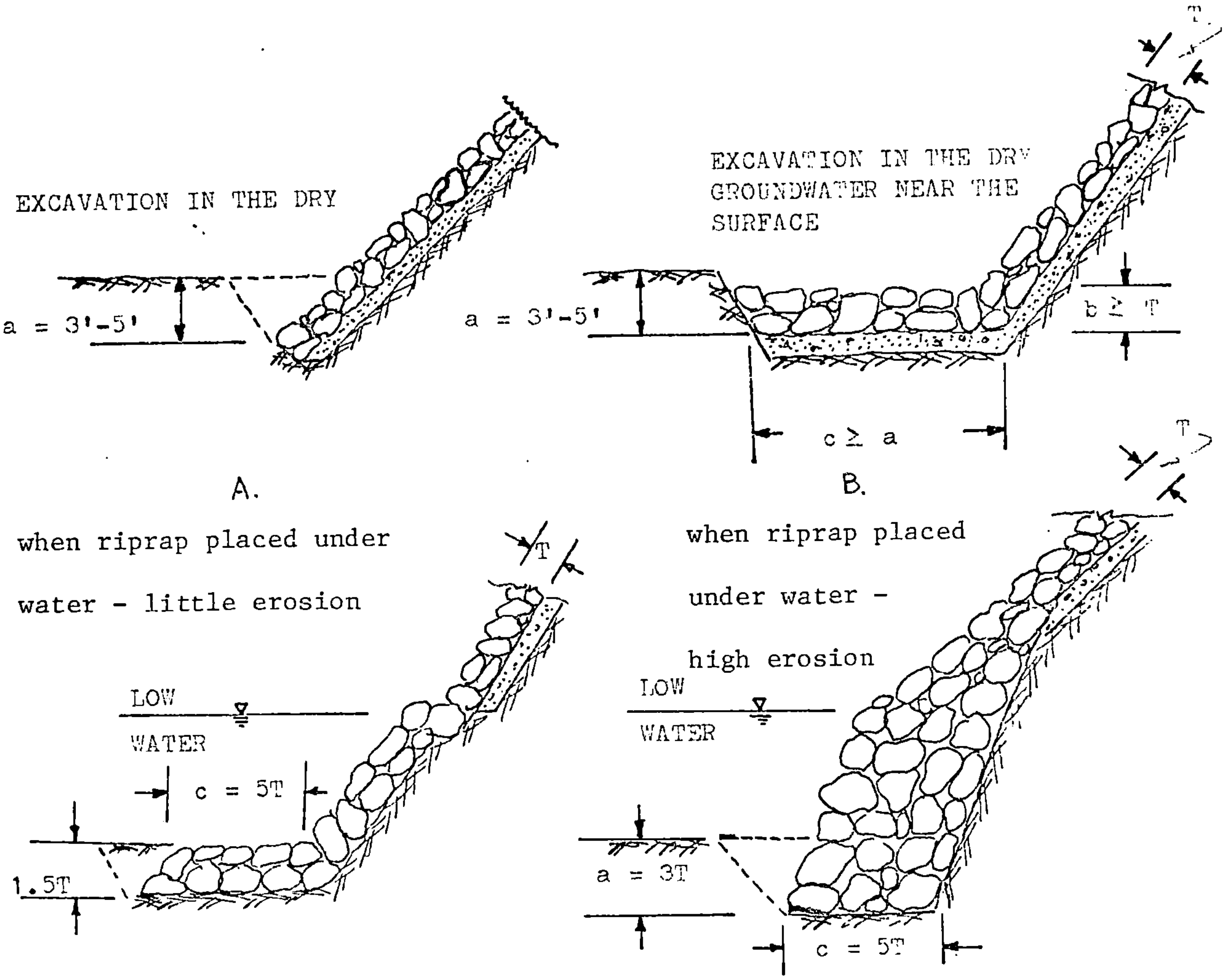
Riprap Linings - Rock, either as riprap or encased in wire can provide an inexpensive and aesthetically

pleasing lining material. Obviously the economic advantage of rock decreases in regions where sound rock is difficult to find.

Riprap failures can usually be attributed to one or more of the following:
1) movement of individual stones by a combination of velocity and turbulence, 2) leaching of the natural bed material through the riprap resulting in slumping and 3) undercutting and raveling of the riprap around the edges and at the end of the riprap blanket. A properly designed channel lining must prevent these modes of failure by: 1) the use of an adequate stone size, 2) adequate riprap gradation and provisions for a properly designed filter blanket and 3) adequate freeboard and proper provisions at the riprap blanket edges. Of particular importance is the downstream termination of the riprap blanket, where the turbulence created by the riprap is likely to develop a scour hole at the end of the blanket. It is suggested that either the thickness of the riprap blanket be doubled at the downstream end to prevent undercutting or that the riprap blanket dip below the natural streambed to the estimated depth of downstream scour.

Channels with stable gradients, or where the gradient has been established with drop structures or check dams often require erosion protection for the banks. A riprap lining is often used for this purpose,

however adequate riprap toe protection must be provided to prevent undercutting of the lining. Several suggested toe protection configurations are shown below (55).



C. RIPRAP TOE PROTECTION D.

There are currently two accepted riprap design methods for channels with gradients of 10 percent or less. The first, an emperical method developed by Anderson, at the University of Minnesota, as a part of the National Cooperative Highway Research Program, is

reported in NCHRP Report 108 (2). The second is a theoretical method derived and laboratory-tested by Stevens, Simons and Lewis at Colorado State University (54). The latter will be presented here since it provides a method to check the stability of existing channels, as well as the design of new channels, and, in general, seems to be a more versatile procedure.

The Stevens, Simons and Lewis method (CSU method) will first be discussed in general and later described in more detail using several examples. For the purpose of this manual the method has been simplified by incorporating several of the equations into nomographs. The simplification introduced by nomographs will undoubtedly reduce the accuracy of the method somewhat, but as will be shown later, the nomographs provide sufficient accuracy for most design work. Appendix B contains the original equations from which the nomographs were derived.

In the CSU method the stability of a particular riprap design is represented by a riprap safety factor, defined as the ratio of moments resisting particle motion to the moments tending to rotate the particle out of the riprap blanket. A safety factor of one denotes incipient motion, and a safety factor greater than one indicates the riprap is secure. From theoretical considerations, the riprap safety factor is shown to be a function of: the side slope angle (θ'), the angle of

repose for the riprap (Φ), the down slope angle of the flow velocity vector (λ), the effective stone diameter (K_m) and the stability number (η). The downslope velocity vector angle (λ) is measured from the horizontal (0 for horizontal flow) and provides for non-horizontal flow conditions such as at bends, constrictions, etc. This term is extremely difficult to estimate and probably of necessity will often be assumed to be zero. This assumption can be compensated for by a more conservative allowable riprap safety factor. In general, a safety factor of 1.5 is recommended.

The stability number (η) is given by:

$$\eta = \frac{21\,\tau_s}{(s_s-1)\gamma K_m} \cong \frac{21\,RS}{(S_s-1)K_m} \qquad \text{(IV-1)}$$

where τ_s = Average tractive force

R = Hydraulic radius

S = Channel friction slope (bottom slope for uniform flow)

S_s = Specific gravity of the stone

K_m = Effective stone size.

The effective stone size is given by:

$$K_m = \left[\frac{\sum_{i=1}^{10} K_i^3}{10}\right]^{1/3} \qquad \text{(IV-2)}$$

$$\text{where } K_i(i=1) = \frac{K_o + K_{10}}{2}$$

$$K_i(i=2) = \frac{K_{10} + K_{20}}{2}$$

$$\vdots$$

$$K_i(i=10) = \frac{K_{90} + K_{100}}{2}$$

and the terms K_0, K_{10}, ... K_{100} are the sieve diameters for which 0 percent, 10 percent . . . 100 percent of the riprap material (by weight) is finer. It was found in the CSU studies that the effective stone size (K_m) was generally between K_{50} and K_{67}.

For very wide channels, that is where $R \cong$ depth, the stability number reduces to a function of the mean velocity. This relationship is reproduced as a nomograph in Fig. IV-3. Note that this nomograph is <u>valid only</u> for wide channels. For other conditions, the stability number must be computed from equation IV-1.

The riprap safety factor for the channel bed is given by the reciprocal of the stability number ($1/\eta$). Generally, the stability of the side slopes will be the critical factor in the overall channel design and the riprap safety factor for the side slopes can be determined from Fig. IV-4. Note that this nomograph is valid only for uniform flow or accelerating flow conditions. Decelerating flow conditions will be covered in Chapter V. Fig. IV-4 assumes horizontal flow ($\lambda = 0$) and the riprap safety factor from this nomograph must be adjusted using Fig. IV-5 if the downslope velocity angle

is not zero. In addition to the nomographs, a channel design will usually require the use of Mannings equation. For rock riprap, the Mannings n value has been found to vary only with mean stone size as follows (2):

$$n = 0.0395\ K_{50}^{1/6} \qquad \text{(IV-3)}$$

<u>Suggested Riprap Characteristics</u>

The following are some of the more important characteristics of riprap as specified by the Corps of Engineers (55).

A.) Stone Shape -

1. blocky, cubical shape with sharp clean edges;
2. not more than 25% of the stones reasonably distributed throughout the gradation shall have a length more than 2.5 times the breadth or thickness;
3. no stone shall have a length exceeding three times its breadth or thickness.

B.) Gradation -

1. lower limit $K_{50} \geq K_m$;
2. upper limit K_{50} as necessary to meet remaining requirements;
3. lower limit $K_{100} \geq 2K_m$;
4. upper limit $K_{100} \leq 5K_m$ or as required by the riprap thickness (see below);
5. lower limit $K_{15} \geq .0625$ x upper limit K_{100};

6. upper limit K_{15} as required to satisfy the criterion for a filter.

C.) Riprap Blanket Thickness -

1. 12 inches minimum;
2. thickness $\geq K_{100}$ or 1.5 upper limit K_{50};
3. increase thickness by 50 percent if placed underwater;
4. increase by 6 in. to 12 in. (with appropriate increase in stone size) when subject to attack by waves or floating debris.

In addition to the above, riprap must be sound, durable and not subject to disintegration due to repeated wetting and drying or freezing and thawing. If the water carries any mineral salts, the rock must not react in a detrimental manner with them.

Riprap Linings at Channel Bends - Flow around a bend in an open channel creates secondary currents which result in a greater tendency for particles to roll out of place than in a straight channel. If the downslope velocity angle associated with these currents is known, the preceding method can be used. However, it is generally quite difficult to estimate these downslope angles, so that an alternate procedure developed by Watts is suggested (52). In this method, the riprap size required to protect the bend is related to two conditions: first, how sharp the bend is (long radius

or short radius) and second, the degree of curvature (60° bend, etc.). The first condition is represented by the parameter V^2/R_d, where R_d is the average radius of the bend on the outside bank (see Fig. IV-6). The second condition is represented by the parameter Δ/Δ_c, where Δ is the internal angle of the bend and Δ_c is the angle defined by the intersection of the centerline and the outside bank as shown in Fig. IV-6. If $\Delta \geq \Delta_c$, the bend is defined to be a long bend and if $\Delta < \Delta_c$, a short bend. The basic equations used in the method are:

$$F'_B = \frac{3V^2}{R_d} + 1 \quad \text{when } \Delta \geq \Delta_c \qquad \text{(IV-4)}$$

$$F_b = 1 + [F'_B - 1) \frac{\Delta}{\Delta_c}] \quad \text{when } \Delta < \Delta_c \qquad \text{(IV-5)}$$

F_B and F'_B are riprap coefficients and the required riprap size to protect a bend is either $K_m \times F_b$ or $K_m \times F'_B$ where K_m is the effective riprap size required to protect a straight channel. Equations IV-4 and IV-5 have been combined and reproduced in the graph shown in Fig. IV-7.

For the case where both the sides and bottom of the channel are protected by riprap and the channel is narrow ($T_w/Y < 10$), the rock size should be increased for all surfaces. For the condition of wide channels ($T_w/Y \geq 10$) where only the banks are protected, usually the larger sized material needs to be placed on the

outside bank. The larger material should extend upstream and downstream from the bend for at least the length of the bend which can be approximated by $R_d \times \Delta$, where Δ is in radians. In very sinuous, high velocity channels where standing waves are likely to be generated, both the inside and outside banks should be protected.

EXAMPLE IV-1

Design a riprap lined channel for a discharge of 500 cfs and a bottom slope of one percent. The channel is to be trapezoidal with a bottom width of 8 feet. A supply of rounded cobbles and gravel, with sizes from 40mm to 600mm, is available. Assume the downslope angle is zero, but compensate by designing for a minimum riprap safety factor of 1.5. From Figure IV-2, the angle of repose ranges from 35° for the gravel to 40° for the largest cobbles. Figure IV-1 indicates a side slope of 3:1 is required to approximately balance the tractive forces on the channel sides and bottom, except for the largest size rock. Therefore, use side slope of 3:1. Using $n = 0.0395\ K_{50}^{1/6}$ and Manning's equation, develop the table shown below for a channel base width of 8 ft. and assumed values of mean rock size.

Table IV-1

K_{50} ft	K_{50} mm	n Eq. IV-3	$\frac{AR^{2/3}}{1.49\ S^{\frac{1}{2}}} \cdot Qn$	A	R	Normal Depth = Y	V fps
0.25	76	.031	104	63.3	2.1	3.45	7.9
0.50	125	.035	117	69.2	2.2	3.65	7.2
1.00	305	.040	134	75.3	2.3	3.85	6.6
1.50	457	.042	141	80.0	2.4	4.00	6.3
2.00	610	.044	148	81.2	2.4	4.05	6.2

This is not a hydraulically wide channel since $R \neq Y$, therefore Eq. IV-2 must be used to compute the stability number (η). Assuming that the effective rock size (K_m) is approximately equal to K_{50} and that the rock has a specific gravity (S_s) equal to 2.65, use Figures IV-1 and IV-5 to establish Table IV-2.

Table IV-2

K_m ft	K_m mm	Φ Fig. IV-2	R Table IV-1 ft.	η Equation IV-2	SF Fig. IV-5	SF* Computed
0.25	76	37	2.1	1.07	.81	0.80
0.50	152	38	2.2	.56	1.33	1.25
1.00	305	39	2.3	.29	1.52	1.68
1.50	457	39	2.4	.20	1.63	1.88
2.00	610	40	2.4	.15	1.71	2.02

*These riprap safety factors were computed for comparison using the CSU method (see Appendix B).

Table IV-2 indicates that an effective diameter (K_m) of one foot will yield a riprap factor of safety of about 1.5 as required. The resulting channel will be trapezoidal with 3:1 side slopes, an 8 ft bottom width and a depth of flow of approximately 3.9 ft at a velocity of 6.6 fps. Approximately 3 ft of freeboard should be provided, of which one foot should be lined. The riprap should have an effective size (K_m) of one foot that is, the gradation should be such that K_m as computed by Eq. IV-3 is about one foot. An adequate filter must be provided as discussed in Chapter 2.

Table IV-2 includes computed values of the riprap safety factor for comparison with those obtained from

the nomograph (Fig. IV-5). It can be seen that there are significant discrepancies, particularly at the high end of the safety factor scale. However, it is felt that the considerable time savings afforded by the nomograph override the possible inaccuracies, particularly when so many of the other factors which go into the design are rough estimates at best. The equations for computing the riprap safety factor are shown in Appendix B of this manual.

EXAMPLE IV-2

For the channel designed in Example IV-1 investigate the effects on the riprap safety factor resulting from downslope velocity components (λ).

Table IV-3

(1) K_m ft	(2) S.F. Example IV-1	(3) λ	(4) Reduction factor Figure IV-5	(5) S.F. (2) x (4)
0.5	1.33	0	1.0	1.33
0.5	1.33	10	0.964	1.28
0.5	1.33	20	0.927	1.23
0.5	1.33	30	0.890	1.18
1.0	1.52	0	1.0	1.52
1.0	1.52	10	0.964	1.47
1.0	1.52	20	0.927	1.41
1.0	1.52	30	0.890	1.35
1.5	1.63	0	1.0	1.63
1.5	1.63	10	0.964	1.57
1.5	1.63	20	0.927	1.51
1.5	1.63	30	0.890	1.45

EXAMPLE IV-3

For the channel designed in Example IV-1 determine the effective riprap size required to protect a bend in the channel with a centerline radius of 64 ft and an internal angle of 10°.

Channel depth = 3.9' (see Table IV-1)

Channel top width = (3.9 x 3) 2 + 8 = 31.4'

$$R_d = 64 + \left(\frac{31.4 + 8}{4}\right) = 73.9'$$

$$\Delta_c = \text{arc cos } \frac{64.0}{73.9} = 30^\circ$$

$\Delta_c = 30^\circ$ and $\Delta/\Delta_c = 0.3$

$$\frac{V^2}{R_d} = \frac{(6.6)^2}{64} = 0.68$$

From Figure IV-7, $F_B = 1.6$

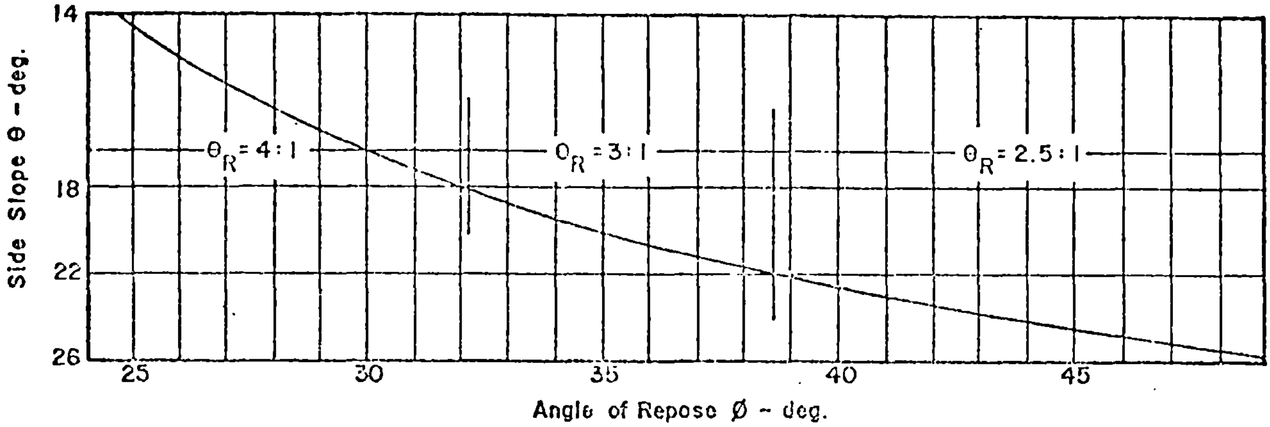
$$\therefore K_B = (1.6)(1.00) = \underline{\underline{1.60'}}\ .$$

Table IV-4

PERMISSIBLE VELOCITY OVER TEMPORARY FLEXIBLE LINING MATERIALS

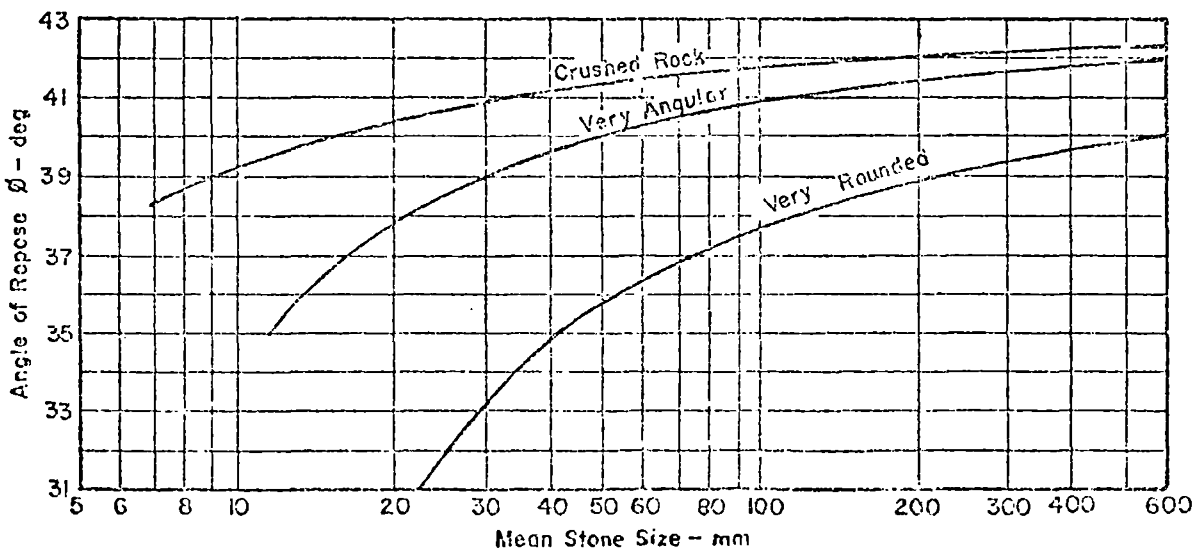
Lining Material	Equation for Maximum Permissible Velocity
Jute Mesh	$V = 61.53\ R^{1.028}\ S_o^{.430}$
Excelsior Mat	$V = 32.29\ R^{1.23}\ S_o^{.351}$
Straw and Erosionet	$V = 70.76\ R^{1.455}\ S_o^{.529}$
Erosionet	$V = 41.45\ R^{.855}\ S_o^{.400}$

From Reference 51.



From Reference 2

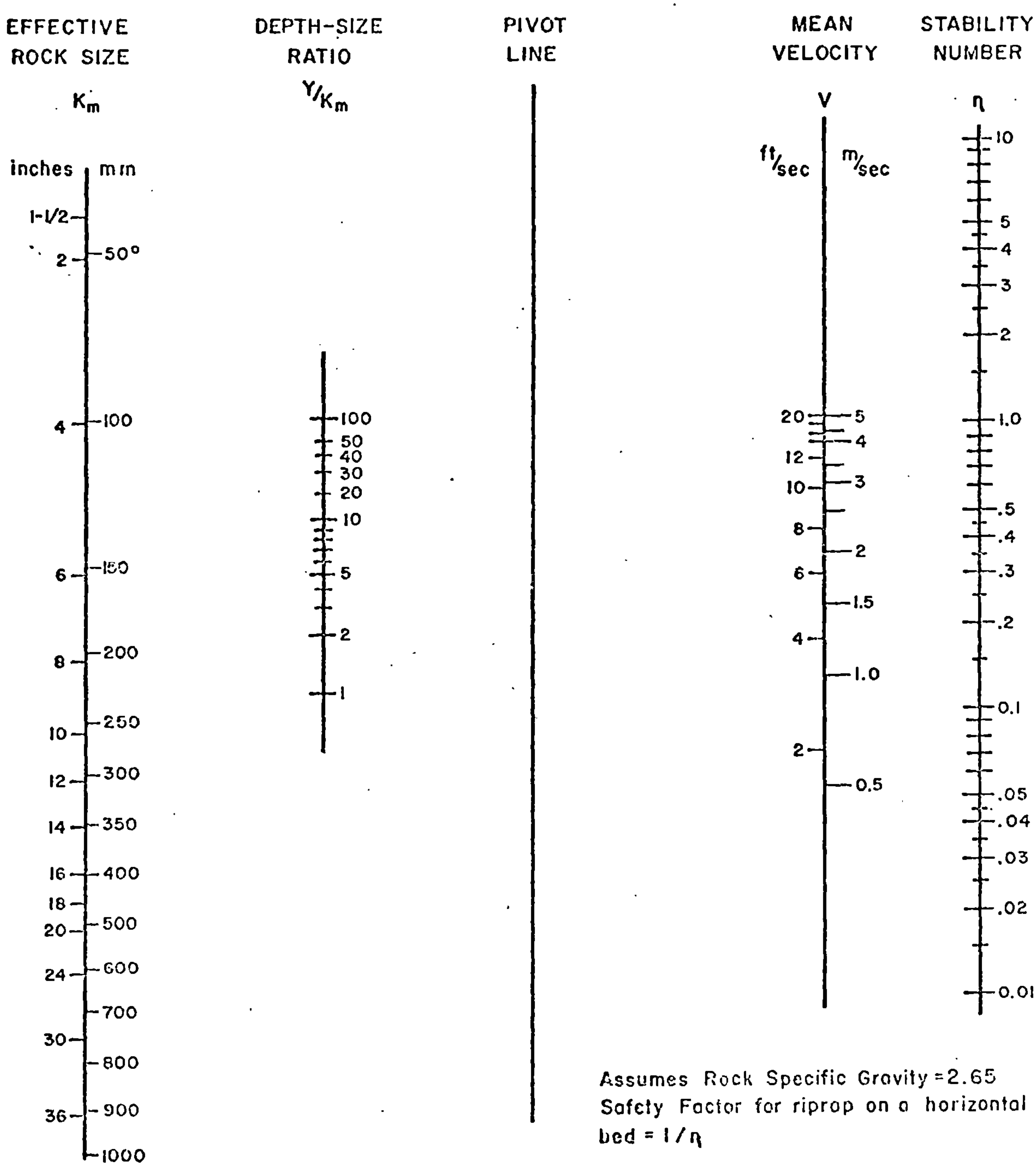
Fig. IV-1



From Reference 2

Fig. IV-2

STABILITY NUMBERS FOR
RIPRAP PROTECTION
with
STEADY-UNIFORM FLOW
IN
WIDE PRISMATIC CHANNELS



Adapted from: "SAFETY FACTORS FOR RIPRAP PROTECTION," Stevens, M.A., Simons, D.B. & Lewis, G.L., CSU Publ. CEP 73-74 MAS-DBS-GLL 68, Colo. State Univ., Ft. Collins, Colo., 1974.

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Fig. IV-3

SAFETY FACTORS FOR RIPRAP PROTECTION
with
Downslope Velocity Angle
(λ) Equal to Zero

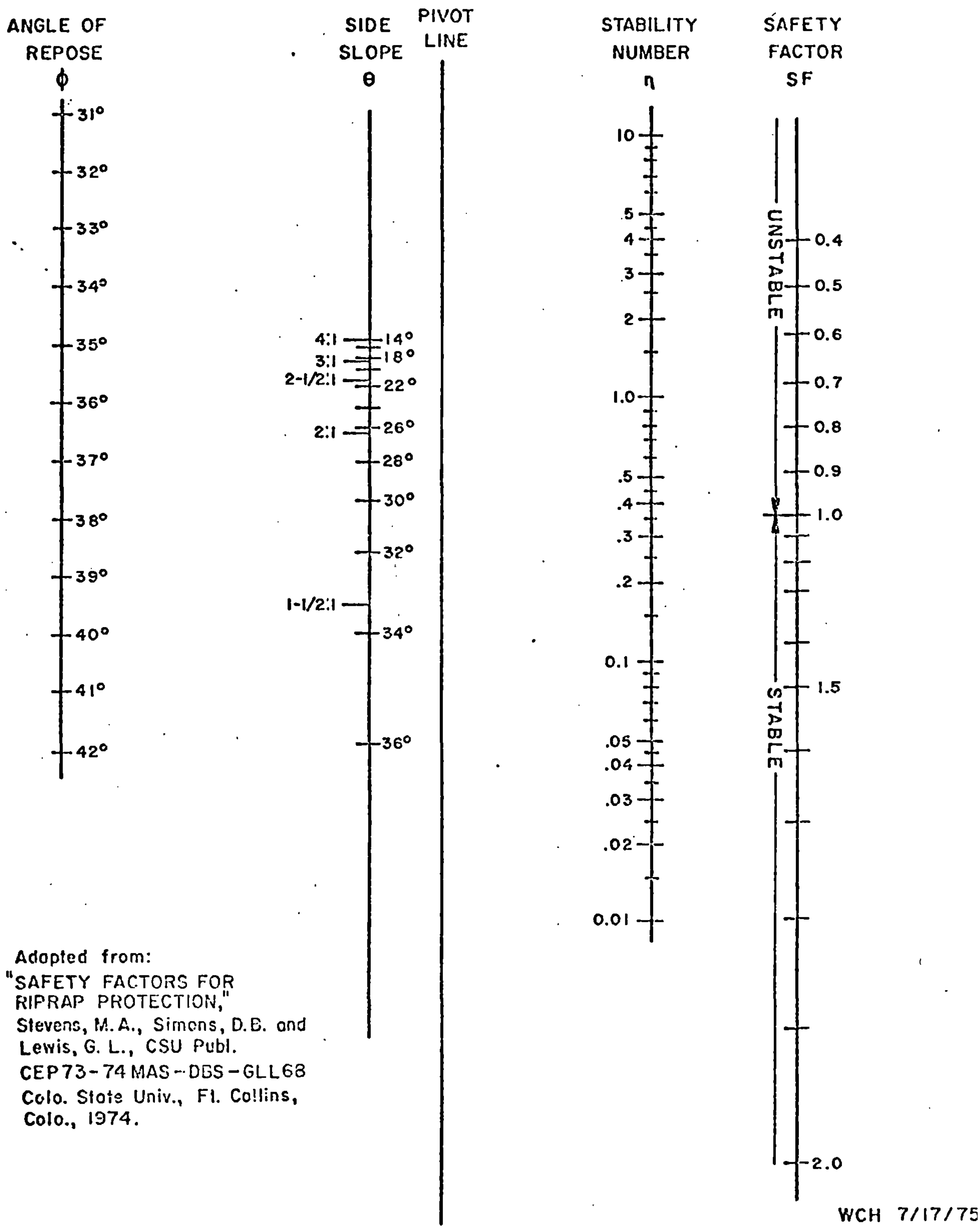


Fig. IV-4

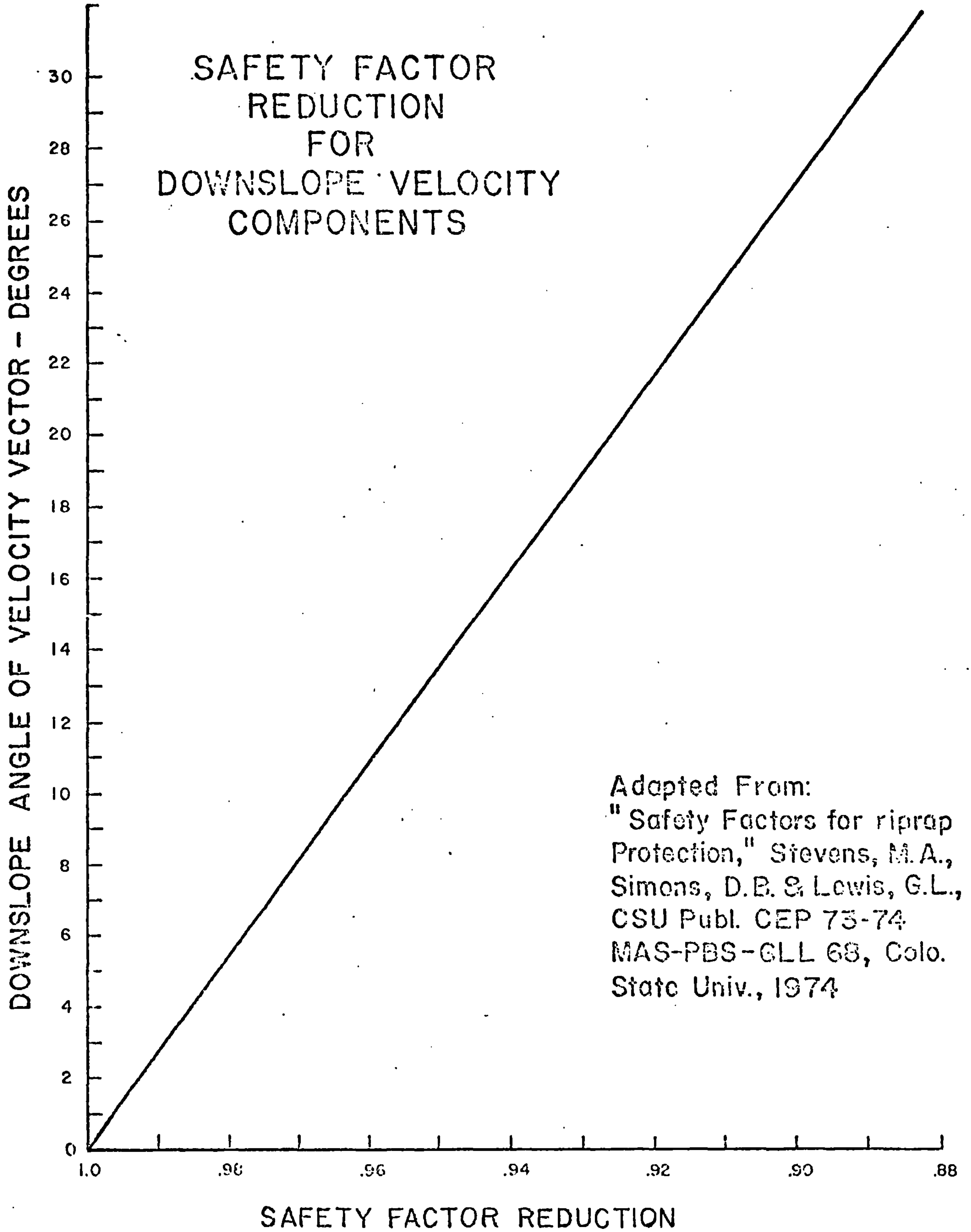
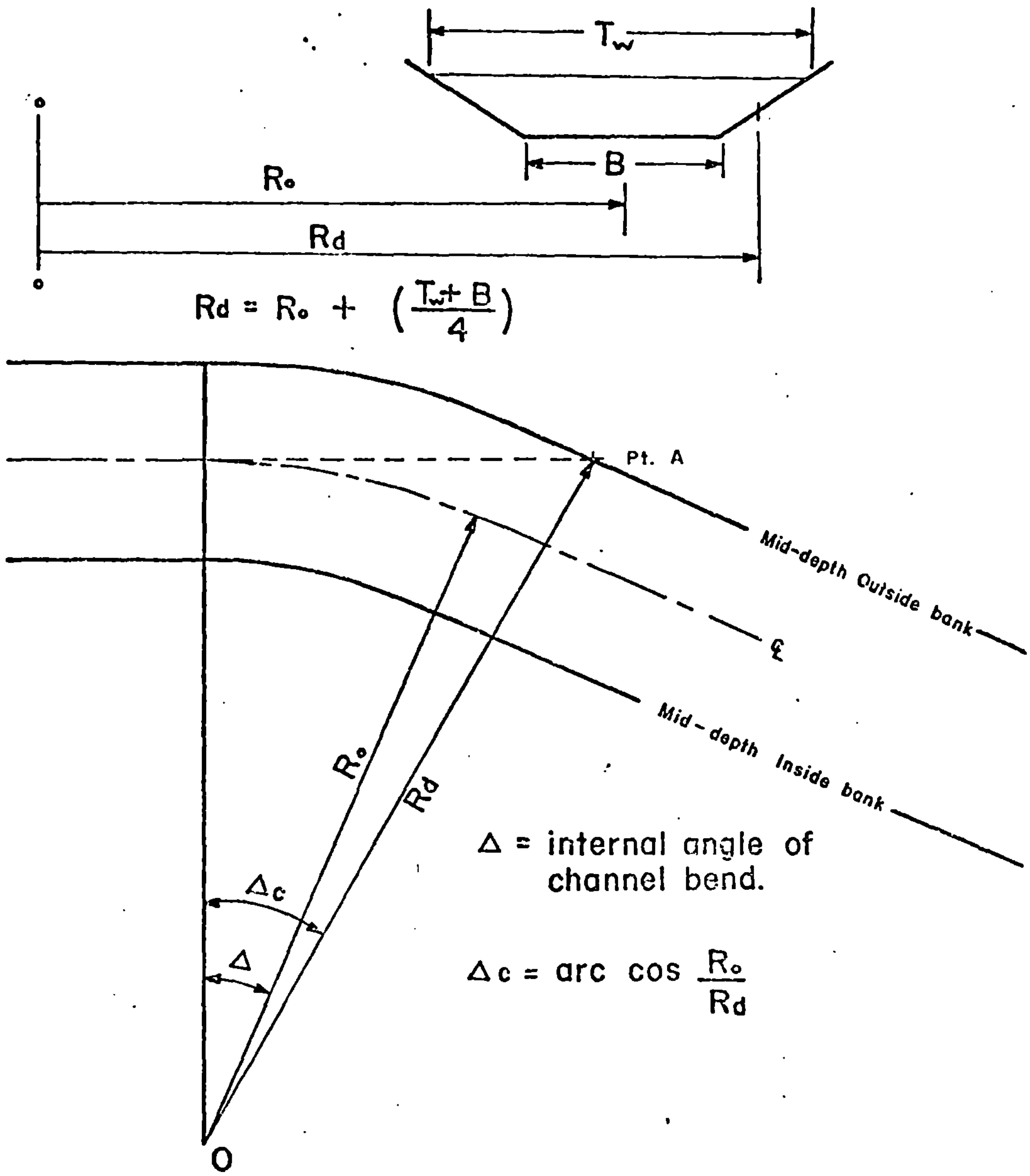

SAFETY FACTOR
REDUCTION
FOR
DOWNSLOPE VELOCITY
COMPONENTS
Adapted From:
"Safety Factors for riprap Protection," Stevens, M.A., Simons, D.B. & Lewis, G.L., CSU Publ. CEP 73-74 MAS-PBS-GLL 68, Colo. State Univ., 1974
DOWNSLOPE ANGLE OF VELOCITY VECTOR - DEGREES
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
1.0
.98
.96
.94
.92
.90
.88
SAFETY FACTOR REDUCTION

Fig. IV-5



VARIABLE DEFINITION FOR CHANNEL BENDS

From Reference 52

Fig. IV-6

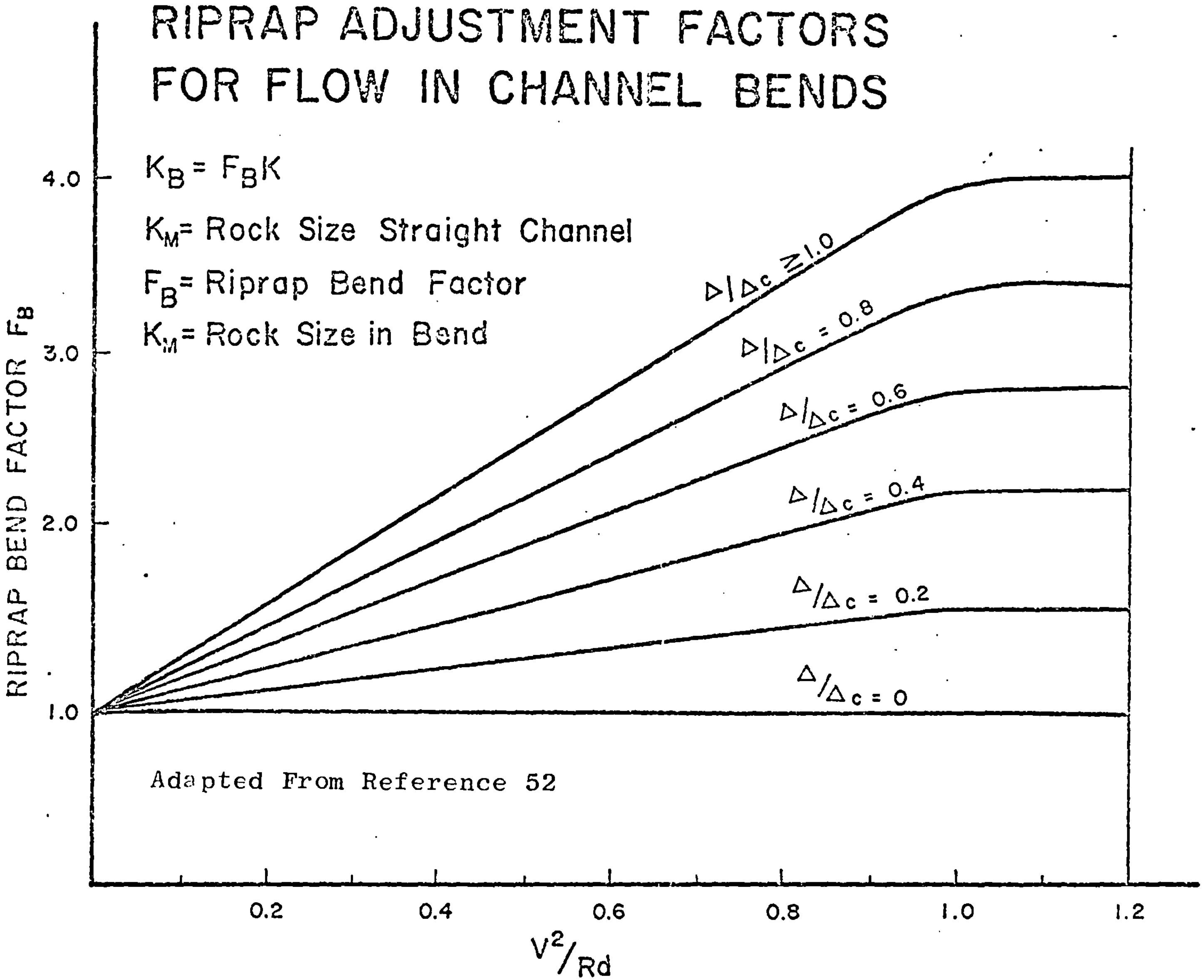

RIPRAP ADJUSTMENT FACTORS
FOR FLOW IN CHANNEL BENDS
K_B = F_B K
K_M = Rock Size Straight Channel
F_B = Riprap Bend Factor
K_M = Rock Size in Bend
Δ/Δc ≥ 1.0
Δ/Δc = 0.8
Δ/Δc = 0.6
Δ/Δc = 0.4
Δ/Δc = 0.2
Δ/Δc = 0
Adapted From Reference 52
RIPRAP BEND FACTOR F_B
4.0
3.0
2.0
1.0
0.2
0.4
0.6
0.8
1.0
1.2
V²/Rd

Fig. IV-7

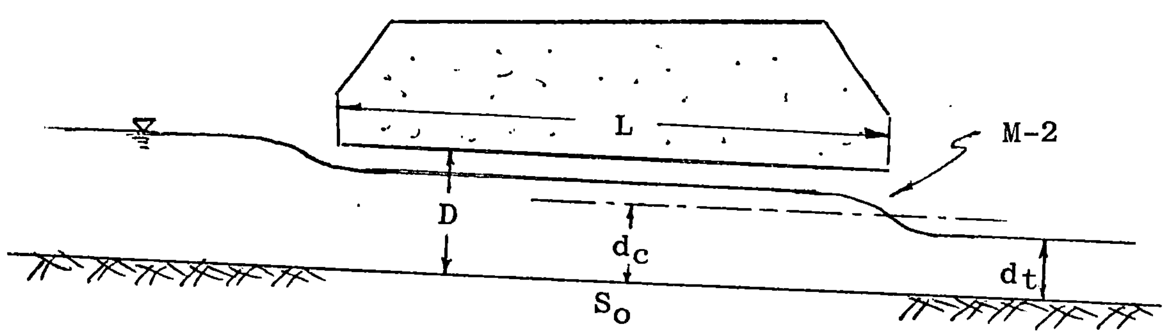
V - SCOUR PROTECTION AT CONDUIT OUTLETS

Scour resulting from highly turbulent decelerating flow is a common problem at conduit and spillway outlets, and can lead to such undesirable consequences as an unstable scour hole, excessive scoured material downstream and possible undermining of the structure itself. Traditionally this problem has been handled with concrete hydraulic jump or impact basins on large structures, and as a maintenance problem with small structures. These solutions are not always acceptable. The concrete structures are expensive and aesthetically unappealing, while treating scour as a maintenance problem does nothing for downstream water quality and might be a matter of too little-too late.

This chapter deals with the design of ripraped scour basins to protect channels below conduit outlets where the flow is characterized by a turbulent plunging jet. Since these flow conditions are not amenable to analytical solution the design procedures presented are emperically derived from model studies. Extensive model studies on ripraped scour aprons and basins have been conducted by the Corps of Engineers (11), Thorston and Shirole, at South Dakota School of Mines, (43), and Simons, Stevens and Watts, at Colorado State University, (40). The design procedure resulting from the Colorado State University study is presented in this manual

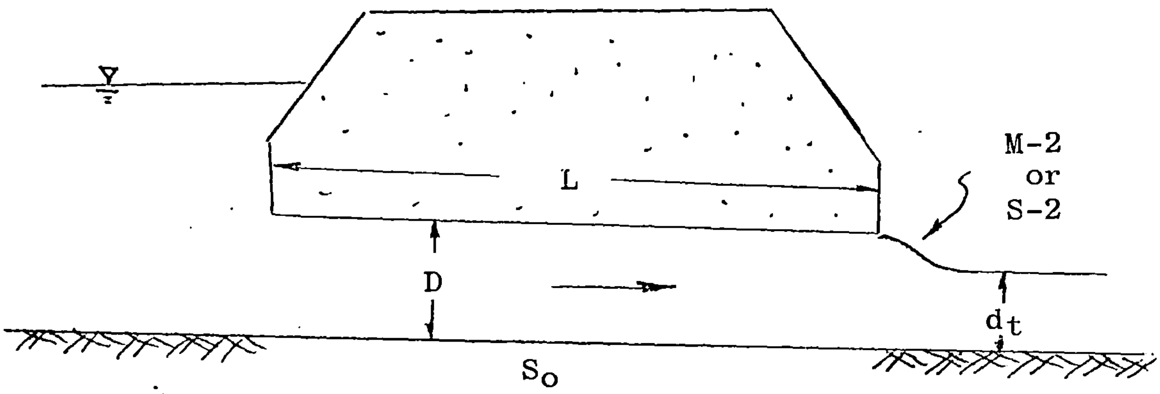
because all the studies yield similar scour basins and the latter is somewhat more flexible. Other scour protection devices for conduit outlets are discussed in Chapter VII.

The Colorado State University method was developed for rectangular and circular conduits with outlet Froude numbers of between 1 and 2.5 and a downward plunging jet at the outlet. The latter is represented by flow conditions which produce M-2 or S-2 flow profiles at the conduit outlet. These conditions generally occur on a mild slope with critical depth at the conduit outlet and a tailwater depth less than critical depth as shown in Fig. V-1A, or for any conduit slope with the pipe flowing full and a tailwater depth less than the vertical height of the conduit as shown in Fig. V-1B. In addition, the method is limited to situations where the culvert slope is parallel with the channel gradient and the conduit outlet invert is flush with the channel.



Mild Slope with Critical
Depth at the Outlet

Figure V-1A



Any Slope with Full Barrel
and d_t Less than D

Figure V-1B

It was found in the model studies that for mild channel slopes with ripraped basins the tailwater elevation at the conduit outlet was approximately equal to the normal depth for the downstream channel. Under these conditions the tailwater will affect the depth of flow at the brink of the conduit outlet. Figures V-6 and V-7 were developed to estimate the conduit brink depth for these conditions.

The design procedure resulting from the model studies was based on the following dimensionless parameters which were found to conserve dynamic similarity over the range of flow conditions studied.

A.) Froude Number -- the Froude number is represented by $Q/D^{2.5}$ for circular conduits, where Q is the design discharge and D is the conduit diameter. The Froude number is represented by $Q/W_oH_o^{3/2}$ for rectangular

conduits, where W_o is the width and H_o the height of the conduit.

B.) d_t/D or d_t/H_o is the degree of submergence where d_t is the tailwater depth.

C.) Y_o/D or Y_o/H_o is the relative depth of flow at the outlet, where Y_o is the depth at the conduit outlet brink depth.

D.) d_s/D or d_s/H_o is the relative depth of scour, where d_s is the depth of scour.

E.) K_m/D or K_m/H_o is the relative rock size, where K_m is the effective particle size as defined in Chapter IV.

The method presented here has been simplified by combining the parameters d_t/D and d_s/D which allowed the design information to be presented on a single graph. The simplification to some extent reduces the accuracy of the method, however it is felt that the simplification is justified in light of the lack of precision inherent in determining the design discharge as well as other hydraulic factors. It is suggested that all rock sizes be increased by 10 percent as a factor of safety. The original CSU curves are provided in Appendix B.

The design procedure results in the selection of one of three standard basin configurations:

A.) <u>The standard non-scouring basin</u> with rock of sufficient size that no scour occurs at flows

equal to or less than the design discharge.

B.) The standard scoured basin in which a controlled scour hole developes. This basin often requires significantly smaller rock than the non-scour basin, but requires a greater volume of rock since the rock must extend below the scour hole which developes, to protect the underlying channel material.

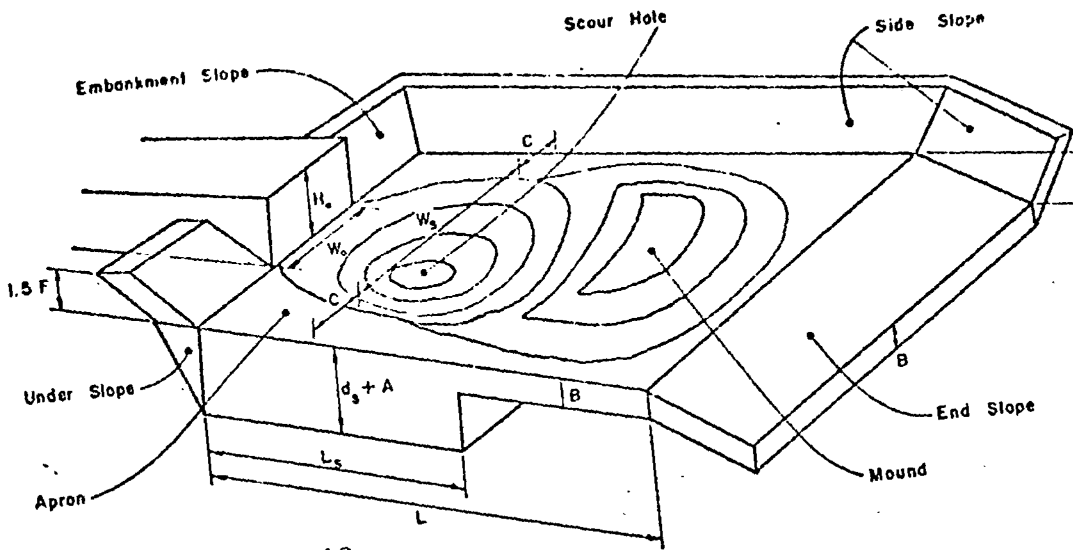
C.) The standard hybrid basin in which some scour occurs but does not rely on a scour hole to dissipate energy as in the scoured basin.

The final selection of basin is based primarily on the cost of obtaining rock. In general, the non-scour basin requires the least volume of rock but requires the largest size of rock, while the scoured basin requires the smallest size rock and the largest volume of rock. Circular culverts with end sections are included in the procedure as a special case of a rectangular conduit.

The configurations of each of the standard basins is shown in Figs. V-2 through V-5. Each standard basin is made up of essentially five components; the apron, the end slope, the side slope lining, the embankment slope lining and the under slope. These components will be discussed in general below and summarized in table form for each standard basin. The volume of rock required for each basin is determined by summing the

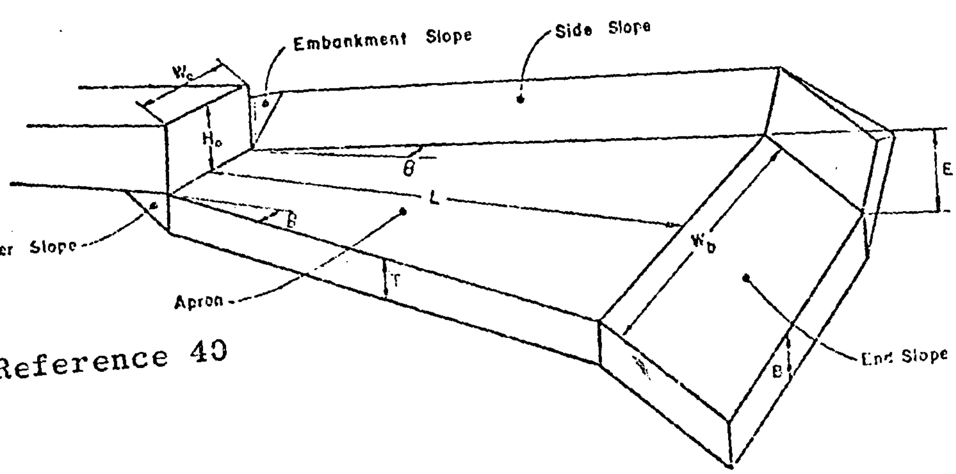
volumes of each component, where each component is assumed to be a prismatic rectangle, triangle or trapezoid.

Referring to Figs. V-2 through V-5, the minimum suggested apron thickness, T, in non-scour basins, is either 2 K_m or K_{100}, whichever is greater. K_{100} is maximum particle size as determined by K_m and the riprap size distribution as discussed in Chapter IV. The minimum thickness of the scoured basins is A plus the depth of scour, d_s, where A is the larger of 2 K_m or K_{100}. The apron flare angle, θ, is determined from Fig. V-13 for the non-scouring basin, or by the required end width, W_b, for the hybrid basin. The end slope terminates the basin and provides protection from local scour which may develop because of the transition from the riprap lining to the natural channel material. If degradation is expected in the downstream channel, the end slope depth, E, should be carried to the anticipated future elevation of the downstream channel bed, otherwise the minimum recommended value for E is the apron thickness. The minimum recommended end slope thickness, B, is K_{100} and the end slope should project into the natural channel at a slope of between 2:1 and 1.5:1. The side slope lining protects the channel banks, extending from the conduit outlet to the termination of the end slope and corresponds approximately to the cross-slope of the natural channel banks. In no



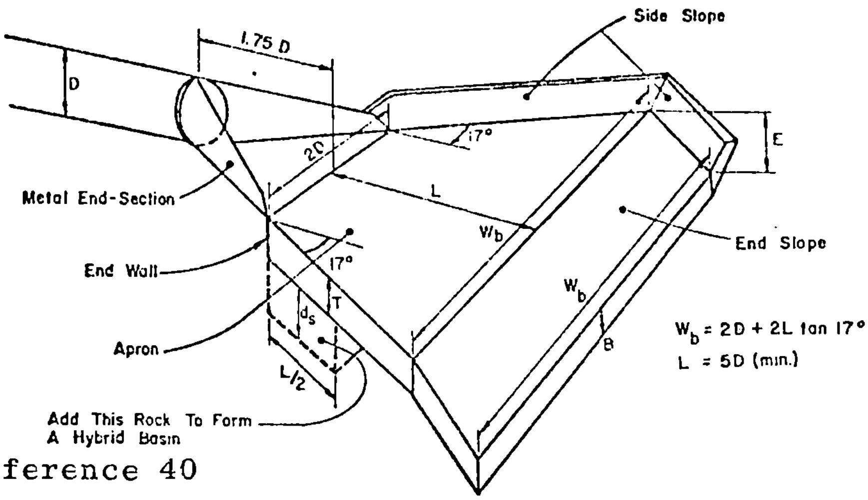
From Reference 40

Fig. V-2



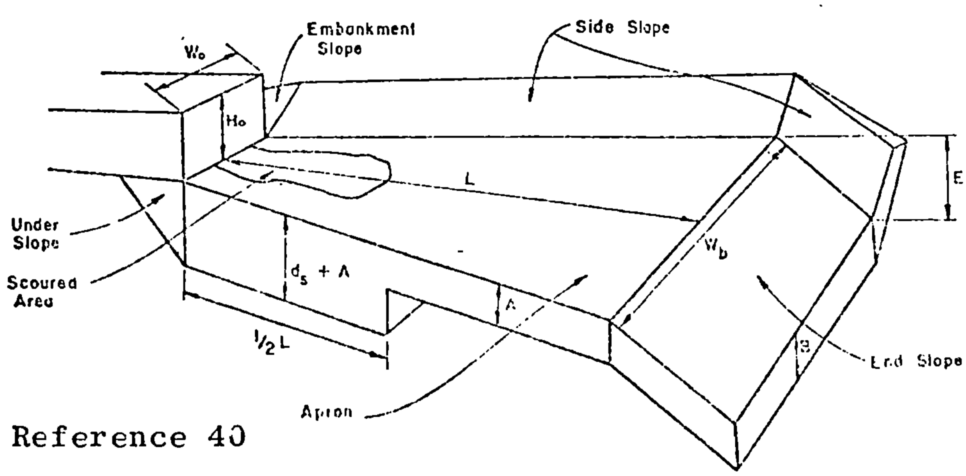
From Reference 40

Fig. V-3



From Reference 40

Fig. V-4



From Reference 40

Fig. V-5

case should the slope be steeper than the angle of repose of the riprap as discussed in Chapter IV. Although it may be possible to use smaller rock on the side slopes, it is suggested that the same size rock be used on the side slopes as in the apron. The minimum suggested thickness of the side slope lining is K_{100} and the vertical height of the side slope lining is 1.5 Y_o or 1.5 d_T, whichever is greater. The embankment slope lining protects the embankment from splash and rollers which may form in the immediate vicinity of the conduit outfall. The embankment slope lining should have the same vertical height and thickness as the side slopes. The under slope joins the embankment slope lining and the apron, and more importantly, prevents the movement of materials out from under the structure. The under slope section is triangular in shape with a vertical height equal to the apron thickness and projects horizontally under the embankment slope lining a distance $B\sqrt{Z_1^2 + 1}$ where Z_1 is the slope of the embankment. The culvert as it joins the basin may be either projecting or mitered. Also, a cutoff wall may be used in lieu of the underslope. If so, it should extend a distance equal to twice the apron thickness (2T or 2A) below the apron.

Standard Non-scouring Basin

The non-scouring basin must be designed so that the high velocity jet at the conduit outlet can expand laterally until the flow velocity is reduced sufficiently to avoid instability in the natural channel. The recommended basin configuration is shown in Fig. V-3, where the length, L, is set so that the mean velocity at the downstream end of the basin is equal to the mean velocity of the downstream channel. The required flare angle is obtained from Fig. V-13. The dimensions of the five components of the basin are summarized in Table V-1.

Table V-1

Basin Component	Minimum Thickness	Remarks
Apron	T = the larger of $2K_m$ or K_{100}	Flare angle from Fig. V-13.
End Slope	$B = K_{100}$	$E_{min} = T$ Slope of end section = 2:1 or 1.5:1.
Side Slope Lining	K_{100}	Vertical height = the larger of 1.5 Y_o or 1.5 d_t
Embankment Slope Lining	K_{100}	Vertical height of embankment slope = vertical height of side slopes.
Under Slope	T	Projects horizontally under the embankment slope a distance $B\cdot\sqrt{Z_1^2 + 1}$ where Z_1 = embankment slope.

Standard Scoured Basin

The standard scoured basin is designed so that the first major flood will dissipate energy by forming a scour hole, piling the material from the scour hole in a mound downstream. In subsequent high flows, flow energy is rapidly dissipated in the boil and roller that

forms in the scour hole. Obviously the design procedure must yield an apron sufficient width, length and thickness to accommodate the scour hole and mound. The recommended configuration is shown in Fig. V-2, where the required apron dimensions are determined from Figs. V-8 through V-12. The dimensions of the five basin components are summarized in Table V-2.

Table V-2

Basin Component	Minimum Thickness	Remarks
Apron	$T = A + d_s$ where A = the larger of $2K_m$ or K_{100}	d_s = depth of scour from Fig. B-13. $L = 1.9\ L_s$ or $2.4\ L_s$ if long term channel degredation is expected (L_s = length of scour hole).
End Slope	$B = K_{100}$	$E_{min} = A$ Slope of end section = 1:1 or 1.5:1.
Side Slope Lining	K_{100}	Vertical height = 1.5 F where F is the larger of Y_o or d_t.
Embankment Slope Lining	K_{100}	Vertical height = 1.5 F.
Under Slope	T	Projects horizontally under the embankment slope a distance $B \cdot \sqrt{Z^2 + 1}$ where Z_1 = embankment slope.

Standard Hybrid Basin

The hybrid basin covers conditions where a basin scours slightly but not enough to yield the efficient energy dissipation associated with the scoured basin. This situation occurs when the ratio of scour depth to effective rock size, d_s/K_m, is between zero and two. That is, when $d_s/K_m = 0$ the basin is non-scouring and when $d_s/K_m \geq 2$ the basin should be the standard scoured basin. Within the range $0 < d_s/K_m \leq 2$ an additional volume of rock is added to the apron of the non-scouring basin to accommodate the scour which occurs, as shown in Fig. V-5. The dimensions of the five basin components, as indicated in Fig. V-5, are shown in Table V-2.

In most cases the design procedure requires the depth of flow and velocity at the brink of the conduit outlet, which must be determined by establishing the water surface profile in the conduit. Where the conduit water surface profile control is the downstream tailwater (mild channels) either Figure V-6 or V-7 may be used to establish the brink depth and velocity. In certain situations the depth and velocity just downstream from the brink of the conduit outlet must be known. The following equations are considered sufficiently accurate for design purposes within two diameters ($2W_o$ rectangular conduit) of the brink if the walls of the downstream channel do not interfere with the spreading of the jet.

$$\left(\frac{V}{V_o}\right)_{ave} = 1.65 - 0.3\ F_o \quad \text{(rectangular conduit)} \qquad \text{(V-1)}$$

$$\left(\frac{V}{V_o}\right)_{ave} = 1.65 - \frac{0.45}{\sqrt{g}}\ \frac{Q}{D^{2.5}} \quad \text{(circular conduit)} \qquad \text{(V-2)}$$

V_o = brink velocity

F_o = brink Froude number

V = average channel velocity within 2D or $2W_o$ of the conduit outlet.

EXAMPLE V-1. Circular conduit on mild slope

Given: A circular culvert with M-2 water surface profile at the outlet. Culvert slope = channel slope. Culvert outlet invert flush with downstream basin.

Design discharge = 110 cfs

48" diameter culvert

Downstream channel is approximately trapezoidal with S_0 = 1 percent, n = .03, 3:1 side slope and a base width of 20 feet.

Find: Determine the required effective rock size, length and width for the three standard ripraped basins.

Assume the tail water depth - normal depth in the downstream channel. At 100 cfs Y_n = 1.0', Y_c = 0.9' and V_n = 4.8 fps.

$$\frac{Q}{D}\,2.5 = \frac{110}{(4)}\,2.5 = 3.44$$

$$\frac{d_t}{D} = \frac{1.0}{4} = .25$$

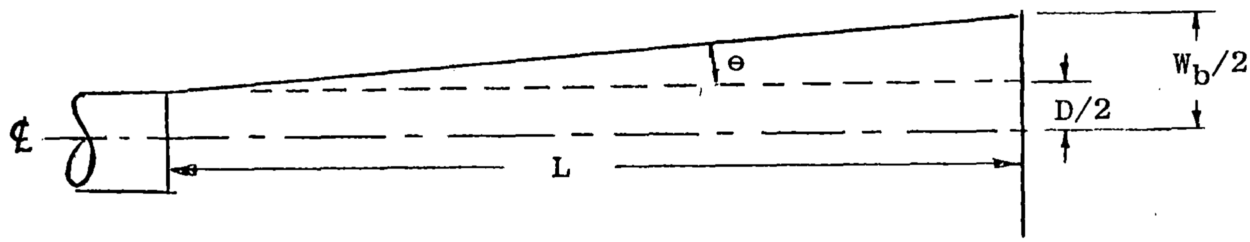
$$\frac{Y_0}{D} = 0.63 \text{ from Figure V-6.}$$

Generate the following table from assumed values of relative depth of scour d_s/D ranging from zero (no scour) to 2, and Figure V-8.

Table V-3 - Rock Size and Expected Scour

(1) d_s/D Assumed	(2) $K_m/D \times d_t/D$ Fig. V-8	(3) K_m/D (2) ÷ 0.25	(4) d_s (1) x 4'	(5) K_m (3) x 4'
0	0.090	0.36	0	1.44
.2	0.060	0.24	0.8	0.96
.8	0.035	0.14	3.2	0.56
1.0	0.029	0.12	4.0	4.48
2.0	0.020	0.08	8.0	0.32

Plot columns 4 and 5 as shown in sketch V-2. The length (l) and width (W_b) of the non-scour basin is established so that the velocity at the basin outlet is the normal channel velocity, or 4.8 fps. The flow cross section of the Jet at the end of the basin is assumed rectangular regardless of the downstream channel shape.



Sketch V-1

The tangent of the flare angle θ from Figure V-13 is 0.5 or θ is about 26° and sin 26° = .45.

$W = D + 2L \sin\theta$ (see sketch V-1) (V-3)

$A = W_b \times Y_{channel}$ (assuming rectangular cross section of flow)

$Q = V_{channel} \times A$

$110 = 4.8 \times 1 \times [4 + 2L(.45)]$

$L = 21'$

$W_b = 23'$

Table V-4 - Width and Length of Scour Hole

(1) d_s/K_m From Table V-3	(2) $\frac{L_s/d_s}{d_t/Y_0}$ Fig. V-11	(3) L_s/d_s	(4) L_s feet	(5) W_s/K_m Fig. V-11	(6) W_s feet
0	32	-	0	0	0
.83	32	12.7	10.2	~10	9.6
5.71	30	12.0	38.4	31	17.4
8.33	24	9.5	38.0	42	20.0
25.00	15	6.0	48.0	94	30.0

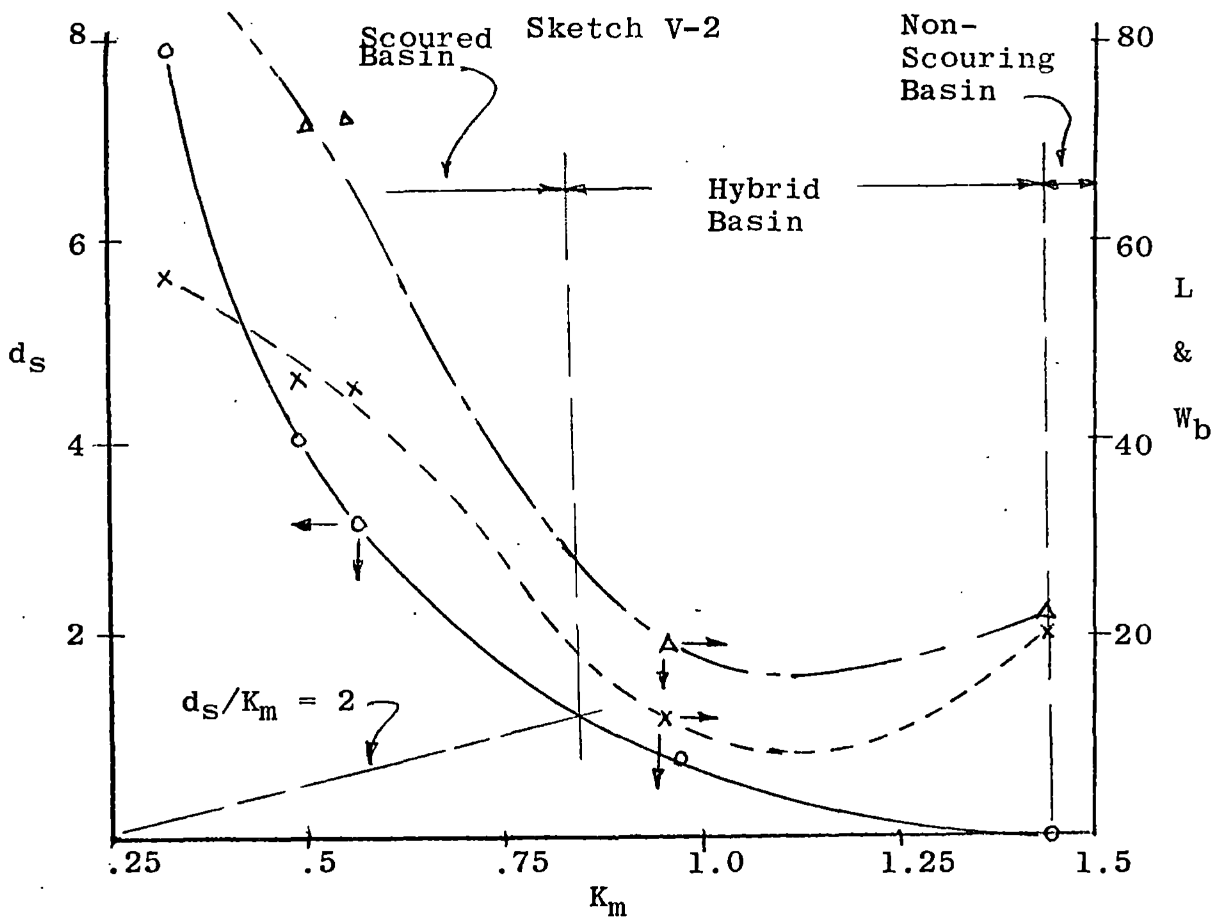
Table V-5 - Width and Length of Basin

d_s/k_m	K_m feet	$L = 1.9L_s$* feet	$W_b = W_s + 2D$ feet
0	1.44	21**	23**
.83	0.96	19.5	18
5.71	0.56	73.0	25
8.33	0.48	72.0	28
25.00	0.32	91.0	38

* If long term channel degradation is expected, use $L = 2.4L_s$.

** Non-scour basin length and width.

Sketch V-2



Plot Table V-5 as shown on Sketch V-2. The lower limit for the hybrid basin (upper limit of non-scouring basin) is determined where $d_s = 0$, that is where $K_m = 1.44$. The upper limit of the hybrid basin is determined where $d_s/k_m = 2$ as shown in Sketch V-2. The final selection of basin is based on the cost of rock, although since the downstream channel base width is 20', it would appear that only the hybrid basin would be a reasonable choice. The scoured basin described in Sketch V-2 is somewhat more conservative than would be obtained using the unmodified curves. These curves are shown in Appendix B.

EXAMPLE V-2. Rectangular Conduit on Mild Slope

Given: A 2.5' x 3.5' concrete conduit flowing full enters the channel described in Example V-1. Design discharge = 110 cfs. M-2 water surface profile at conduit outlet.

Find: Determine the required effective rock size, length and width for the three standard basins.

$$\frac{Q}{W_0 H_0^{3/2}} = \frac{110}{(3.5)(2.5)^{3/2}} = 7.95$$

$$\frac{d_t}{H_0} = \frac{1}{2.5} = 0.4$$

$$\frac{Y_0}{H_0} = 1.0 \text{ (conduit full)}$$

Table V-6 - Rock Size and Expected Scour

d_s/H_0 Assumed	$K_m/H_0 \times d_t/H_0$ Fig. V-9	K_m/H_0	d_s feet	K_m feet
0	.21	.53	0	1.33
.2	.15	.38	.5	1.00
.8	.12	.30	2.0	.75
1.0	.11	.28	2.5	.70
2.0	.057	.14	5.0	.35

$\tan\theta = .5$ from Figure V-13 ($\theta = 26°$)

$110 = 4.8 \times 1 \times [3.5 + 2L(.45)]$

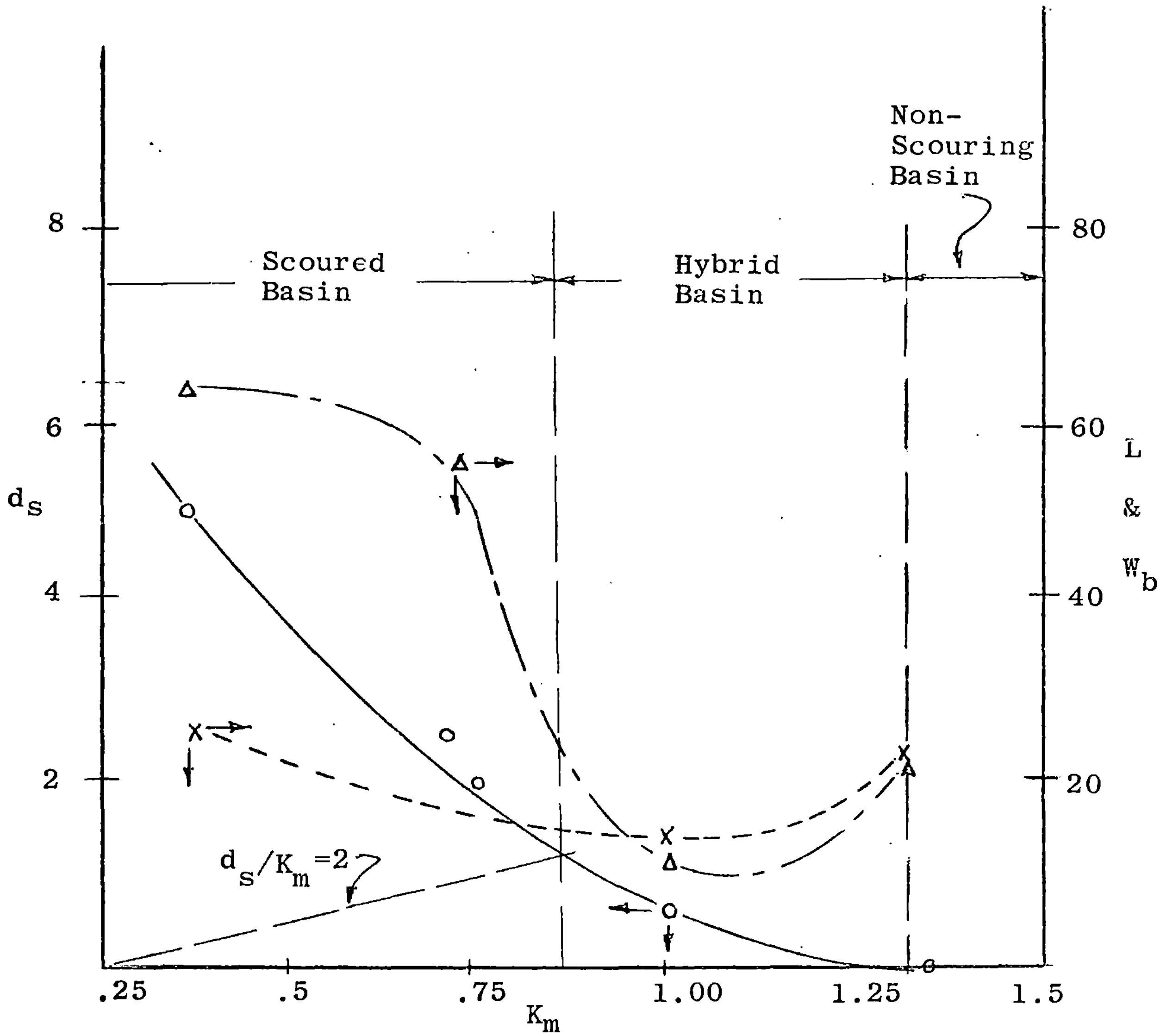
$$\left.\begin{array}{l} L = 22' \\ W_b = 23' \end{array}\right\} \text{non-scoured basin}$$

Table V-7 - Length and Width of Basin

d_s/K_m Table V-6	$\frac{L_s/d_s}{d_t/Y_0}$ Fig. V-11	L_s/d_s	L_s	W_s/K_m	W_s	K_m feet	$L = 1.9L_s$	$W_b = W_s + 2H_0$
0	-	-	0	-	-	1.33	22*	23*
.5	32	12.8	6.4	10	10	1.00	12	15
2.7	32	12.8	25.6	17.5	13	.75	49	18
3.6	32	12.8	32.0	22	15	.70	61	20
14.3	16.5	6.6	33.0	61	21	.35	63	26

*Non-scouring basin length and width.

Sketch V-3



Non-
Scouring
Basin
Scoured
Basin
Hybrid
Basin
8
6
4
2
80
60
40
20
d_s
L
&
W_b
$d_s/K_m=2$
.25
.5
.75
1.00
1.25
1.5
K_m

EXAMPLE V-3. Circular Conduit on Steep Slope

Given: Circular culvert and channel have steep slope.

Culvert outlet invert is flush with downstream basin.

Design discharge = 142 cfs

Downstream channel is approximately trapezoidal with S_0 = 4 percent, n = .03, 3:1 side slopes and a base width of 20 feet.

48" diameter culvert

Culvert operates under inlet control with an outlet brink depth of 2.4'.

Find: Determine the required effective rock size, length and width for the three standard rip-raped basins.

For the given conditions normal depth in the downstream channel is 0.8 feet with a velocity of 7.9 fps.

Then $d_t = 0.8'$

$Y_0 = 2.4'$

$Y_0/D = 0.6$ from hydraulic characteristics of circular sections a/A = .626

$$V_0 = 142/(.626)\left[\frac{\pi(4)^2}{4}\right] = 18.1 \text{ fps}$$

Supercritical flow is parallel to culvert at outlet while subcritical flow shows significant curvature several feet (diameters) ahead of outlet (M-2 water surface profile). However, a pipe flowing full under

subcritical conditions runs parallel to pipe similar to supercritical conditions. The method suggested for handling conduits on steep slopes is to convert the outfall flow conditions of the steep sloping conduit into equivalent conditions for mildly sloping conduit flowing full. This involves finding equivalent flow parameters for a conduit flowing full with same outlet velocity (V_0), tailwater depth (d_t) and brink depth (Y_0) as actual conduit.

Let $Y_0 = D_{equiv} = 2.4' \therefore Y_0/D = 1$

$V_0 = 18.1$ fps

$d_t = 0.8$

$d_t/D = 0.8/2.4 = .33$

$(Q/D^{2.5})_{equiv} = \left[(18.1) \times \frac{\pi}{4}(2.4)\right] / (2.4)^{2.5} = 9.17$

Table V-8 - Rock Size and Expected Scour

d_s/D	$K_m/D \times d_t/D$	K_m/D	d_s	K_m
0	.31	.93	0	2.23
0.6	.24	.72	1.44	1.73
1.0	.15	.45	2.40	1.08
2.0	.078	.23	4.80	.55

The jet expansion angle from Figure V-13 is not valid for steep slopes, but can be estimated using the following relationship:

$$(\tan\,\theta)_{steep} = \tan\,\theta \left[\frac{F_o}{(F_o)_{equiv}} \right] \qquad \text{(V-4)}$$

where tan θ comes from Figure V-13

F_0 = actual Froude Number

$(F_0)_{equiv}$ = Froude Number for equivalent conditions

Note that tan θ can never be less than .07.

$Q/D^{2.5} = 142/4^{2.5} = 4.44$

$(\tan\,\theta)_{steep} = (.38)\,\frac{4.44}{9.17} = .184$

$\theta = 10.4^{\circ}$ and $\sin\,\theta = .18$

Length and width of non-scouring basin from

$Q = V_{channel} \times Y_{channel}\,[D + 2L\,\sin\,\theta]$

$142 = 7.9 \times .8\,[4 + 2L(.18)]$

$L = 51.3$ feet

$W_b = 4 \times 2(51.3)(.18) = 22.5$ feet

Table V-9 - Width and Length of Basin

ds/Km	$\frac{L_s/ds}{d_t\ y_0}$	L_s/ds	L_s	W_s/Km	W_s	Km	$L = 1.9L_2$	$W_b = W_s + 2D$*
0	-	-	0	-	0	2.23	51.3**	22.5**
.83	36	11.88	17.1	10+	17.3	1.73	32.5	25.3
2.22	36	11.88	28.5	17	18.4	1.08	54.2	26.4
8.73	26.6	8.78	42.1	43	23.7	.55	80.0	32.0

* Note: Use actual conduit diameter.

** Non-scouring basin length and width.

EXAMPLE V-4. Circular Conduit with End Section

Given: A 36" culvert with end section carries 55 cfs at a 1 percent slope. The downstream channel is trapezoidal with 3:1 side slopes, n = .03, 15' base width and gradient of .01.

Find: Determine the required effective rock size, length and width for the three standard basins. Normal depth in the downstream channel is 0.8 feet with an associated velocity of 4 fps.

$$\frac{Q}{D^{2.5}} = \frac{55}{(3)^{2.5}} = 3.52$$

$$\frac{d_t}{D} = \frac{0.8}{3} = .27$$

NOTE: The suggested upper limit for applying this method to conduits with end sections is $\frac{Q}{D^{2.5}} = 3.5$ and $\frac{d_t}{D} = .33$. Therefore this example just meets the suggested requirements.

From Figure V-6

$$\frac{Y_0}{D} = 0.64.$$

It is possible that the tailwater could produce a hydraulic jump within the culvert end section, which would drastically change the flow conditions at the end section exit. Therefore this possibility must be explored using the momentum equation, $\beta \frac{Q^2}{g\ A} + \beta'\bar{Z}A$ = constant, where β is assumed equal to one and β' is equal

to 0.65 for closed conduits (For a more complete discussion see chapter VII). Using the flow characteristics of a circular section and $\frac{Y_0}{D} = .64$, yields $\frac{A_0}{A} = .67$ and $\bar{Z} \cong .6'$.

Also, $A_0 = (.67)\left[\frac{\pi(3)^2}{4}\right] = 4.7\ ft^2$ and $V_0 = \frac{55}{4.7} =$ 11.6 fps. Specific force at the conduit brink using the momentum equation is

$$\frac{(1)(55)^2}{(32.2)(4.7)} + (.65)(.6)(4.7) = 21.7\ ft^3.$$

Assuming a rectangular section for the end section, the sequent depth at the end section exit (2D wide) is obtained from the momentum equation,

$$\frac{(55)^2}{(32.2)(6)Y} + (\frac{Y}{2})(6Y) = 21.7\ ft^3.$$

Solving by successive approximation yields a sequent depth of 2.2 feet, i.e., if a hydraulic jump forms, the tailwater must be at least 2.2 feet. Since the tailwater depth is well below 2.2' (tailwater = 0.8') a hydraulic jump will not form in the end section and the flow eminating from the end section will display the characteristics appropriate for the three standard basins.

The suggested method for establishing the effective rock size and basin dimensions is to treat the end section as an equivalent rectangular conduit and assume the exit depth just equals the height of the equivalent rectangular culvert so that $y_0/H_0 = 1.0$. This method

requires the use of Eq. V-2 to establish the depth and velocity at the exit from the end section.

$$\left(\frac{V}{11.6}\right)_{ave} = 1.65 - \frac{.45}{\sqrt{32.2}}\ (3.52)$$

or V = 15.5 fps is the exit velocity and $Y = \frac{55}{(15.9)(6)} =$ 0.6' is the exit depth. Since the exit depth from Eq. V-2 is less than the tail water depth, use 0.8' as the depth (exit depth - tailwater depth if depth from Eq. V-2 is less than tailwater depth) and therefore the exit velocity is $\frac{55}{6(.8)} = 11.5$ fps.

$$\frac{Q}{W_0 H_0^{3/2}} = \frac{55}{(6)(.8)^{1.5}} = 12.8$$

$$\frac{d_t}{H_0} = 1$$

$$\frac{y_0}{H_0} = 1.$$

The remainder of the procedure is exactly as shown in Example V-2.

EXAMPLE V-5. Multiple Conduits

Given: Three 2.5' x 3.5' concrete conduits flowing full carry a combined discharge of 330 cfs. The conduits empty into a trapezoidal channel which flows at a depth of one foot and velocity of 4.8 fps.

Find: The required effective rock size and dimensions for the three standard basins.

The design of rock basins for multiple-barrel culverts is essentially the same as for a single barrel if all the conduits are the same size. The scour depth and length will be the same as for a single conduit discharging Q_t/n. The width of the scour hole is given by:

$$W_{sn} = W_s + (n-1)(W_0 + T) \qquad \text{(V-5)}$$

or

$$W_{sn} = W_s + (n-1)(D + T) \qquad \text{(V-6)}$$

where

W_{sn} = width of the scour hole for n conduits

W_s = width of the scour hole for one conduit

n = number of conduits

T = spacing between conduits.

In this example $Q/n = \frac{330}{3} = 110$ cfs.

$$\frac{Q}{W_0 H_0^{3/2}} = \frac{110}{(3.5)(2.5)^{3/2}} = 7.95$$

$$\frac{d_t}{H_0} = 0.4$$

$$\frac{y_0}{H_0} = 1.0$$

These parameters are the same as those used in Example V-2. Lengths and depths of scour versus rock size would be the same as those shown in Tables V-6 and V-7. If the spacing between conduit is 1.5 feet, then the width of the scour hole would be given by:

$$W_{sn} = W_s + (2)(3.5 + 1.5) = W_s + 10$$

The required basin width would be given by:

$$W_{bn} = W_{sn} + 2H_0 = W_s + 10 + 2(2.5)$$

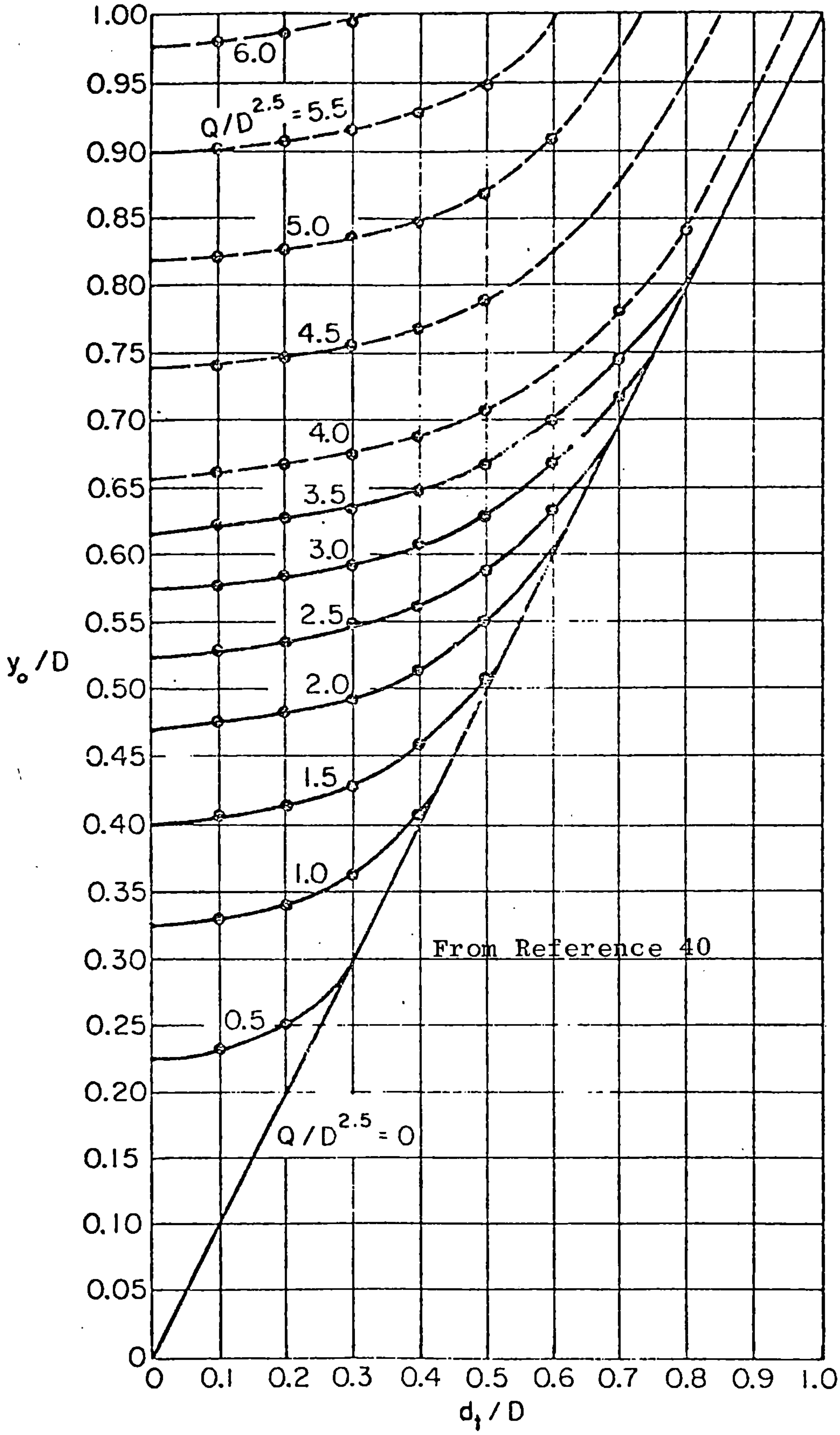
$$W_{bn} = W_s + 15.$$

Thus, for an effective rock size of one foot, the depth of scour is 0.5' (Table V-6), the length of scour is 6.4' (Table V-7) and the width of scour is:

$$W_{sn} = 10 + 10 + 20 \text{ feet.}$$

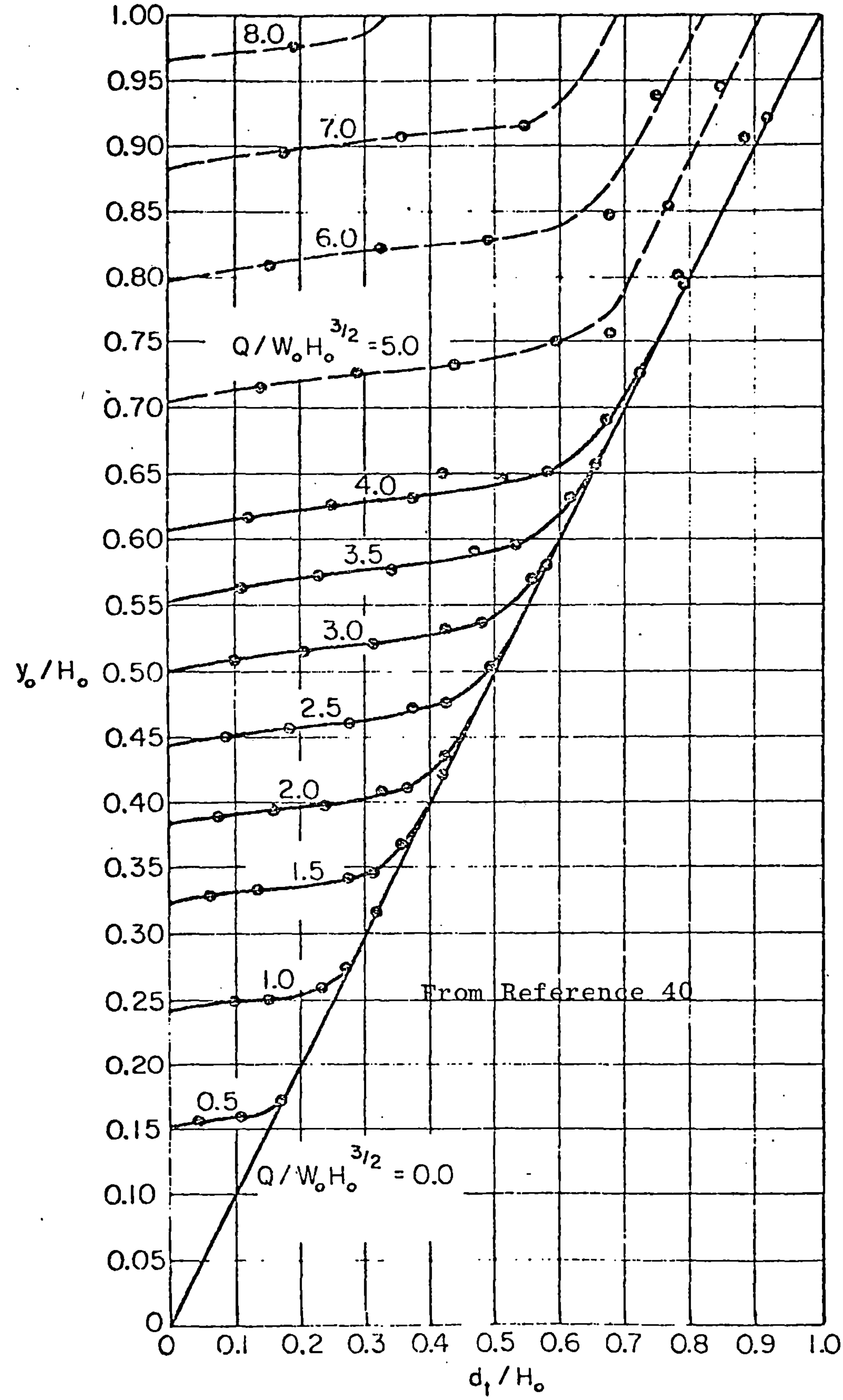
For an effective rock size one foot, the basin length is 12 feet (Table V-7) and the basin width is:

$$W_{bn} = 10 + 15 = 25 \text{ feet.}$$



Effect of Tailwater on Brink Depth: Horizontal and Mild Sloping Circular Culverts

Fig. V-6



Effect of Tailwater on Brink Depth: Horizontal and Mild Sloping Rectangular Culverts

Fig. V-7

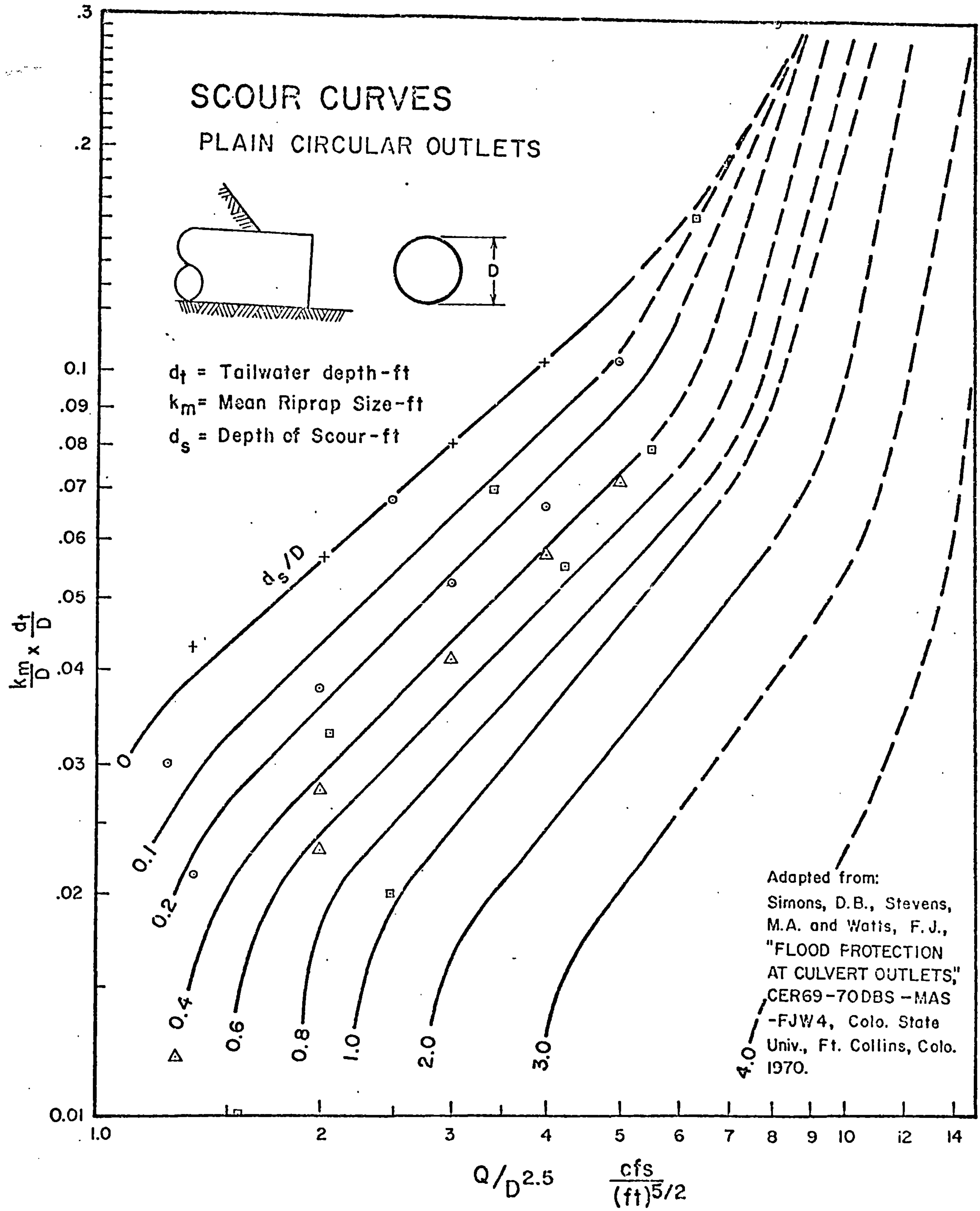

SCOUR CURVES
PLAIN CIRCULAR OUTLETS
D
d_t = Tailwater depth-ft
k_m= Mean Riprap Size-ft
d_s = Depth of Scour-ft
d_s/D
0
0.1
0.2
0.4
0.6
0.8
1.0
2.0
3.0
4.0
$\frac{k_m}{D} \times \frac{d_t}{D}$
.3
.2
0.1
.09
.08
.07
.06
.05
.04
.03
.02
0.01
1.0
2
3
4
5
6
7
8
9
10
12
14
$Q/D^{2.5}$ $\frac{cfs}{(ft)^{5/2}}$
Adapted from:
Simons, D.B., Stevens,
M.A. and Watts, F.J.,
"FLOOD PROTECTION
AT CULVERT OUTLETS,"
CER69-70DBS-MAS
-FJW4, Colo. State
Univ., Ft. Collins, Colo.
1970.

Fig. V-8

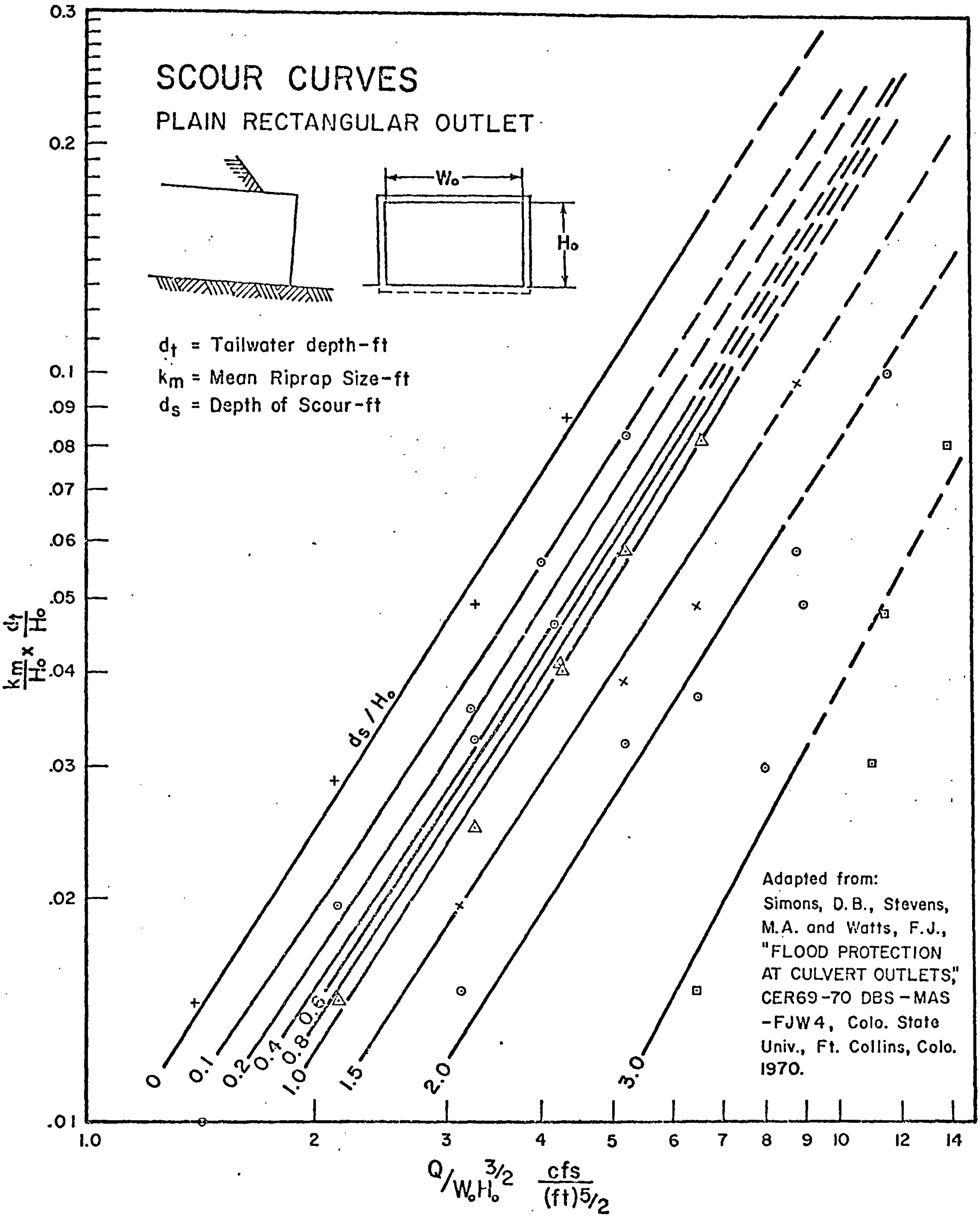

SCOUR CURVES
PLAIN RECTANGULAR OUTLET
Wo
Ho
dt = Tailwater depth-ft
km = Mean Riprap Size-ft
ds = Depth of Scour-ft
ds / Ho
0
0.1
0.2
0.4
0.6
0.8
1.0
1.5
2.0
3.0
km/Ho x dt/Ho
Q/WoHo^3/2 cfs/(ft)^5/2
0.3
0.2
0.1
.09
.08
.07
.06
.05
.04
.03
.02
.01
1.0
2
3
4
5
6
7
8
9
10
12
14
Adapted from:
Simons, D.B., Stevens, M.A. and Watts, F.J., "FLOOD PROTECTION AT CULVERT OUTLETS," CER69-70 DBS-MAS-FJW4, Colo. State Univ., Ft. Collins, Colo. 1970.

Fig. V-9

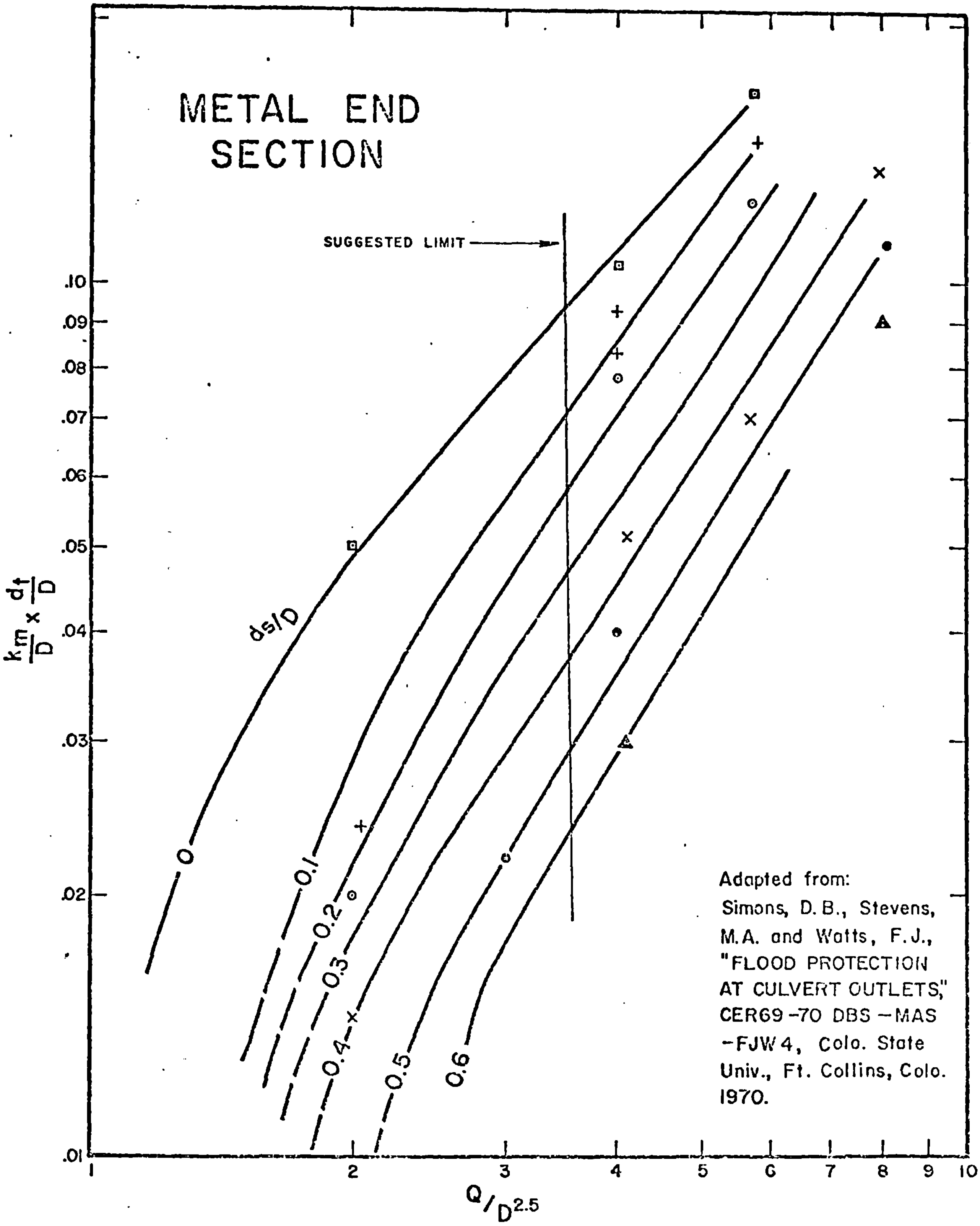

METAL END
SECTION
SUGGESTED LIMIT
ds/D
0
0.1
0.2
0.3
0.4
0.5
0.6
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01
1
2
3
4
5
6
7
8
9
10
$\frac{k_m}{D} \times \frac{d_t}{D}$
$Q/D^{2.5}$
Adapted from:
Simons, D. B., Stevens,
M.A. and Watts, F.J.,
"FLOOD PROTECTION
AT CULVERT OUTLETS,"
CER69-70 DBS-MAS
-FJW 4, Colo. State
Univ., Ft. Collins, Colo.
1970.

Fig. V-10

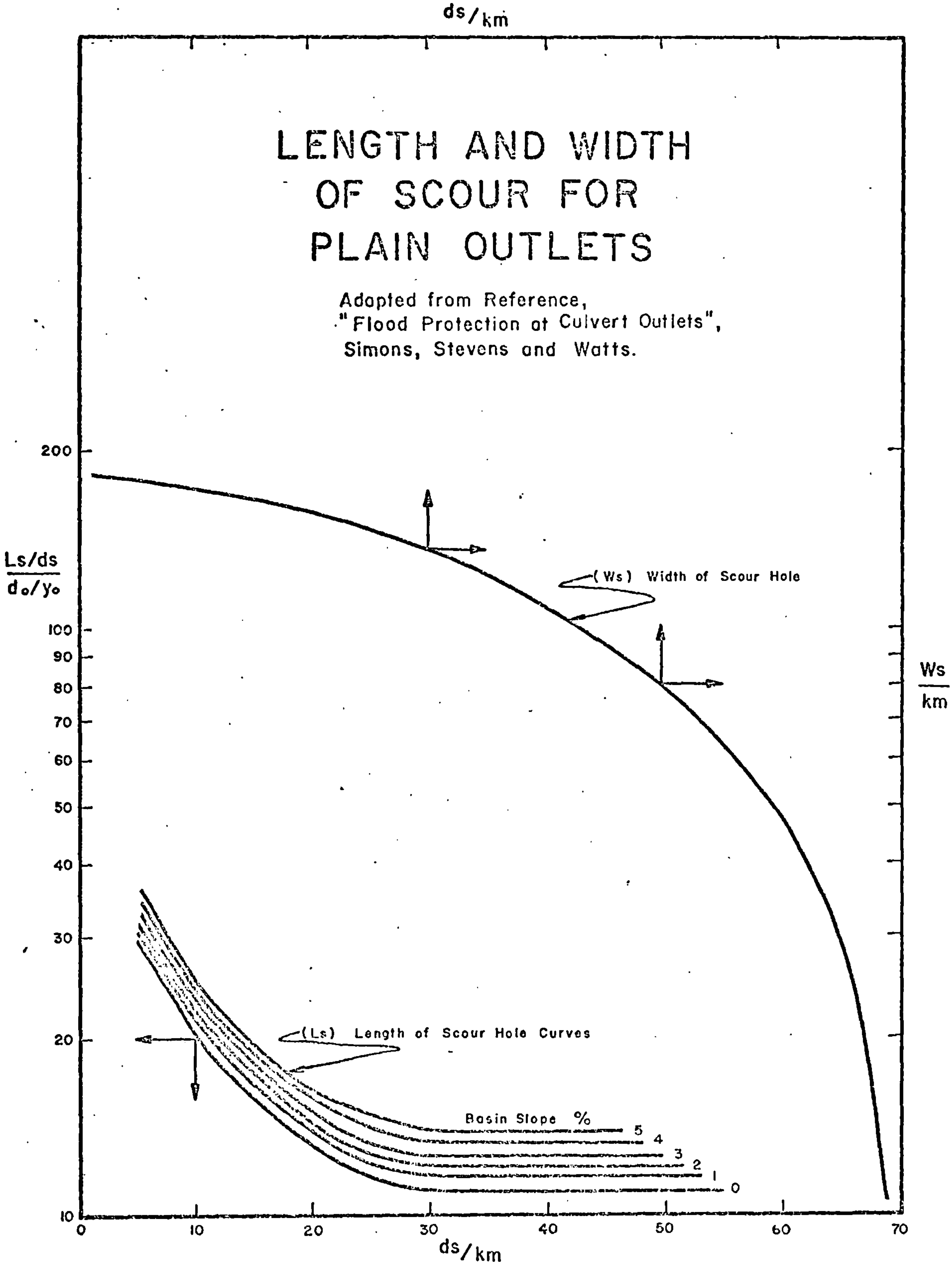

ds/km
LENGTH AND WIDTH
OF SCOUR FOR
PLAIN OUTLETS
Adapted from Reference,
"Flood Protection at Culvert Outlets",
Simons, Stevens and Watts.
Ls/ds
do/yo
200
100
90
80
70
60
50
40
30
20
10
(Ws) Width of Scour Hole
Ws
km
(Ls) Length of Scour Hole Curves
Basin Slope %
5
4
3
2
1
0
0
10
20
30
40
50
60
70
ds/km

Fig. V-11

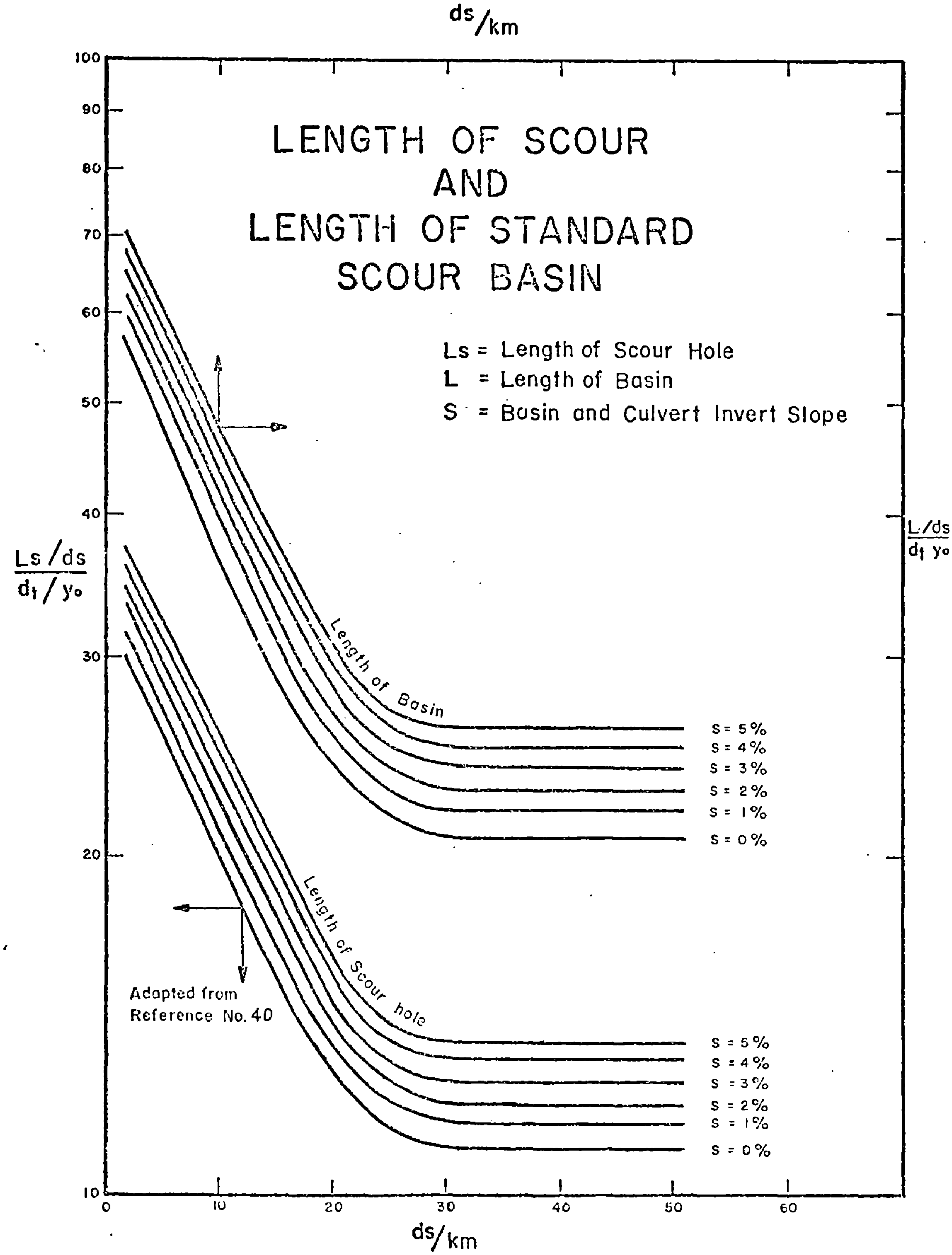

ds/km
LENGTH OF SCOUR
AND
LENGTH OF STANDARD
SCOUR BASIN
Ls = Length of Scour Hole
L = Length of Basin
S = Basin and Culvert Invert Slope
Ls/ds
dt/yo
L/ds
dt yo
Length of Basin
Length of Scour hole
S = 5%
S = 4%
S = 3%
S = 2%
S = 1%
S = 0%
S = 5%
S = 4%
S = 3%
S = 2%
S = 1%
S = 0%
Adapted from
Reference No. 40
100
90
80
70
60
50
40
30
20
10
0
10
20
30
40
50
60
ds/km

Fig. V-12

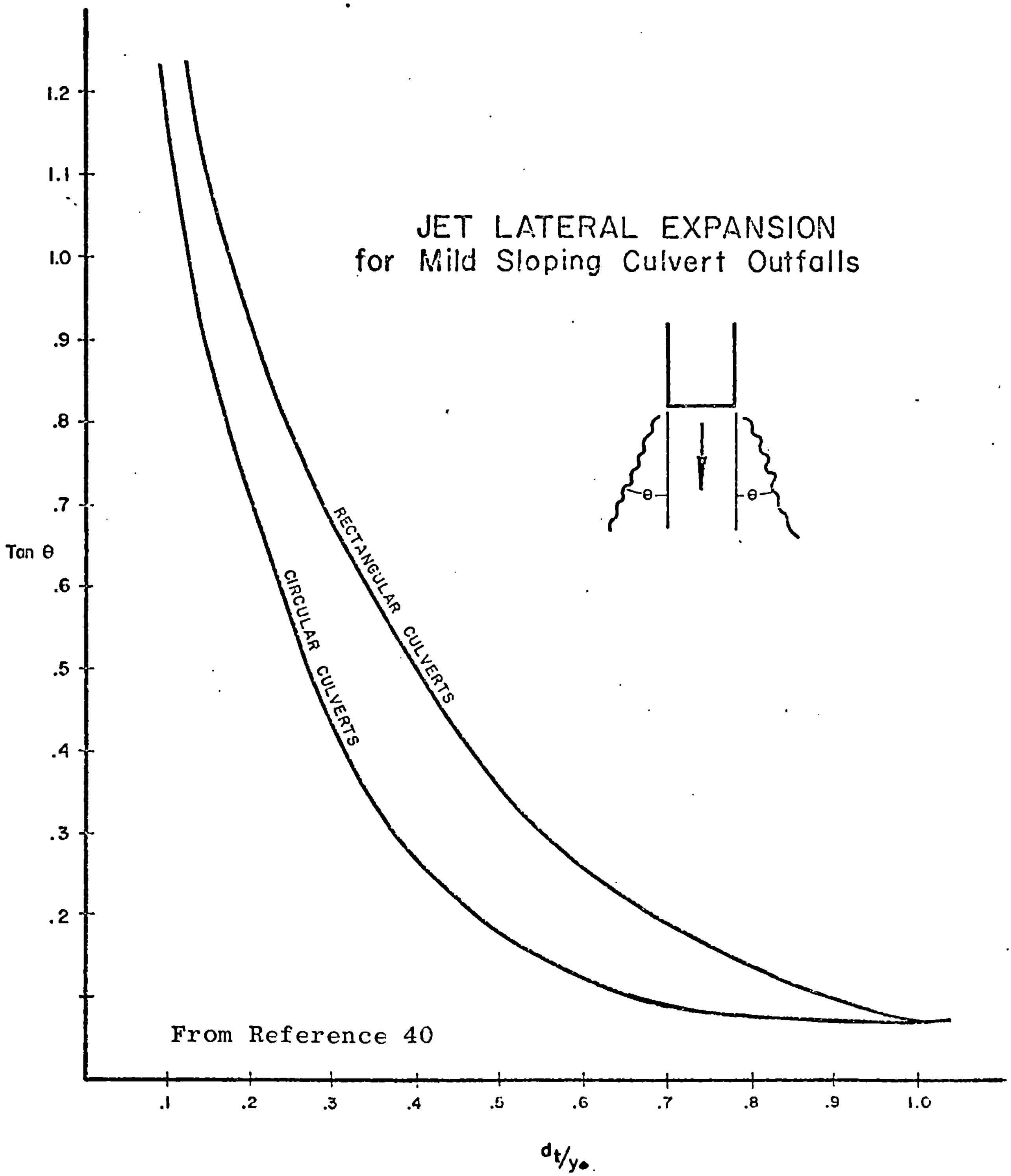

JET LATERAL EXPANSION
for Mild Sloping Culvert Outfalls
Tan θ
1.2
1.1
1.0
.9
.8
.7
.6
.5
.4
.3
.2
RECTANGULAR CULVERTS
CIRCULAR CULVERTS
θ
From Reference 40
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
d_t/y_*

Fig. V-13

VI - WIRE ENCASED ROCK

Wire encased rock refers to rocks that are bound together with wire so that they act as a single unit. In essence it amounts to making a large rock by binding many small rocks together with wire, and in fact, this is one of the major advantages of wire encased rock - it provides an alternative in situations where available rock sizes are too small for dumped riprap. Another advantage is the versatility that results from the regular geometric shapes of wire encased rock. Rectangular blocks, rectangular mats and cylinders can be fashioned into almost any shape that can be formed with concrete. It is far more flexible than concrete, yet maintains its structural integrity better than dumped riprap. Wire encased rock, like riprap, is porous and, thus, not subject to uplift pressures. Again, like riprap, it should be founded on an appropriate filter to prevent leaching of the bed material. As stated in Chapter IV, channels with bottom slopes greater than or equal to 10 percent should be lined with wire encased rock rather than dumped riprap. This rather arbitrary rule stems first, from the feeling that at slopes of about this degree the required size of dumped riprap becomes uneconomically large and second, that the structural integrity of wire bound rock is far superior to dumped riprap - even using stones two to three times larger.

The durability of wire encased rock is generally limited by the service life of the galvarized binding wire which, under normal conditions, is considered to be about 15 years. Water carrying silt can be very abrasive and reduce the service life of the wire; also water which rolls, or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil effect and considerably shortens the life of the wire. The latter can be partially overcome by capping key points - weir lips, drop structure aprons, etc. - with concrete; also providing toe protection for wire encased rock side slopes with dumped riprap has proven to be an effective protection. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified.

Wire encased rock is not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while still relatively small, they may be patched by weaving new strans of wire into the wire cage. In any case, a design should account for possible wire failure, either by a regular maintenance program or by arranging the wire encased rock so that failure of the wire will not result in a complete failure of the structure.

Wire baskets for encasing rock are commercially available in two basic configurations (50)(57). The revetment mattress, a basket made from 14 guage wire is 6'6" wide and comes in lengths of 8', 10' and 12', and heights of 6" or 9". The revetment mattress is intended primarily for lining channels. The gabion, a rectangular basket made from 11 guage wire is 3' wide and is available in 6', 9' and 12' lengths and heights of 12", 18" and 36". The size, shape and mass of the latter is more suited for situations where the structure must serve as a retaining wall as well as erosion protection.

Another configuration which is not commercially available is a cylindrical wire basket, often referred to as a "rock sausage", which is formed with cylindrically wound wire (as used for chain link fences) and when laid parallel to the flow, provides a stable channel lining, even in steep channels. The cylindrical wirebound rock, or "rock sausage", is not as versatile as the rectangular shape but has the advantage that it has a somewhat more manageable size and weight. The latter makes it possible to manufacture the cylindrically wirebound rock at an off-site construction yard and then haul them to the site by truck. The finished rock cylinders are then rolled or lifted into place and tied together with wire.

Although bound rock for erosion protection and channel stabilization was used by the early Egyptians, surprisingly little design information is available. The Maccaferri Gabion Technical Handbook (57) and Bekaert Gabion Handbook (50) provide a number of examples of successful channel stabilization configurations and some general design rules. These design rules, which are summarized below, should be considered rules of thumb. Whether these design rules are used or not, the design should include a check to determine whether each wire encased unit which makes up the channel protection structure is secure from movement due to impinging water and, as stated previously, founded on an adequate filter.

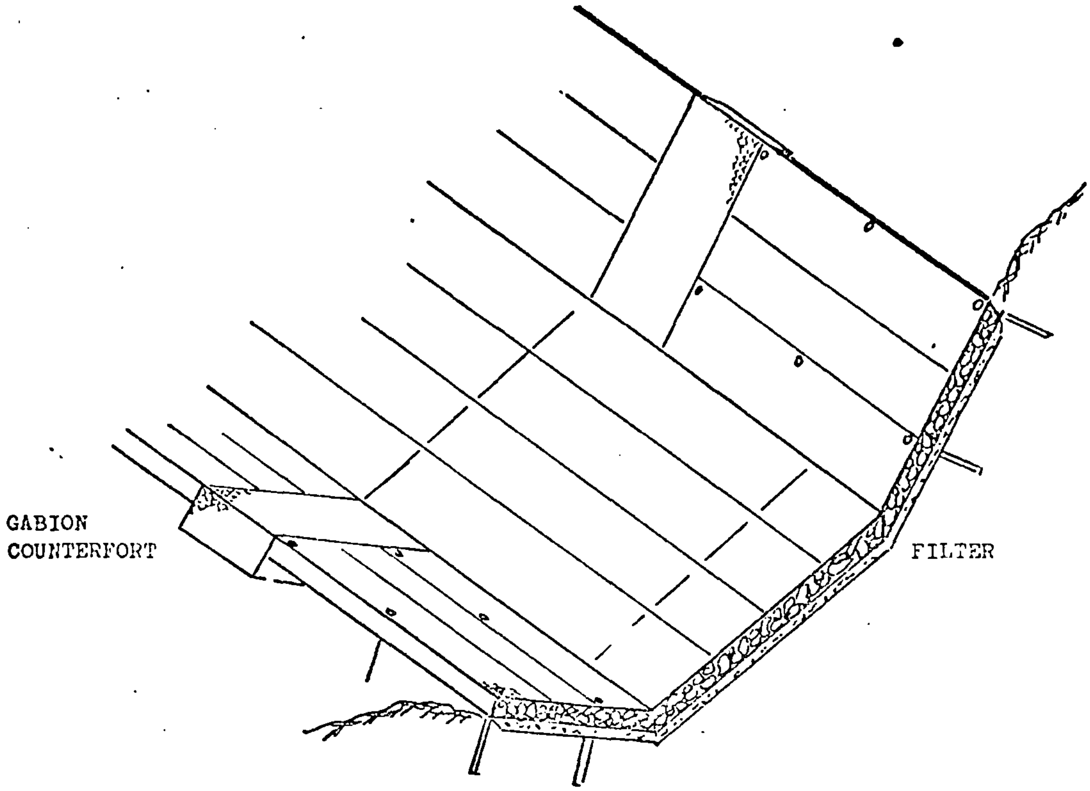
Suggested Design Rules

A.) Channel Linings -- uniform channels with low to moderate turbulence and mean velocity less than 10 fps.

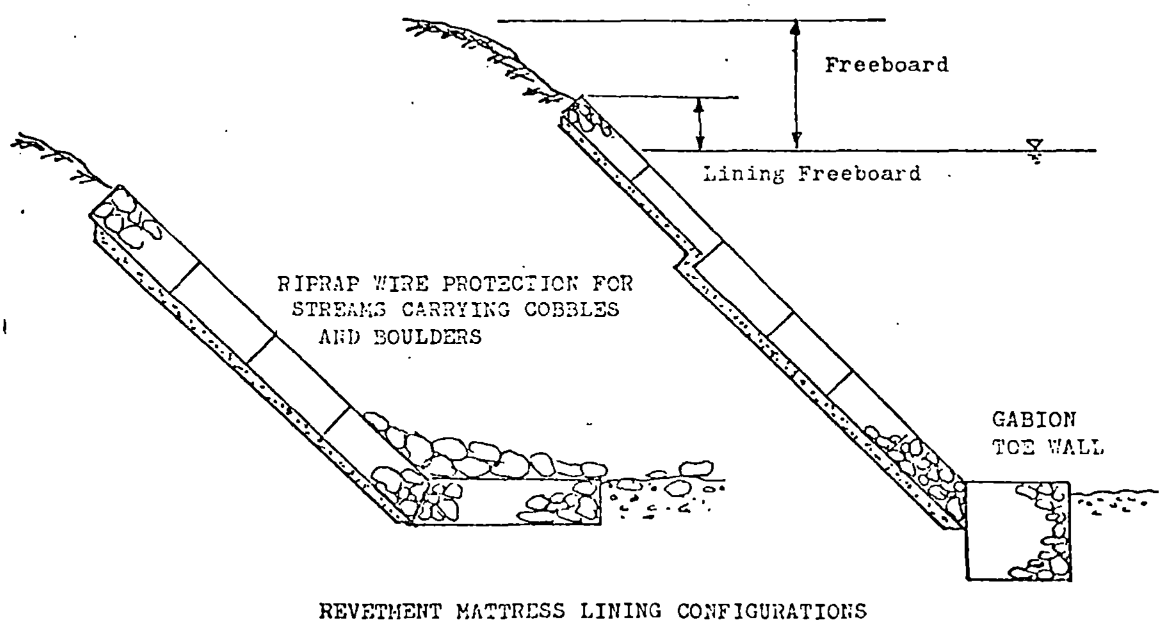
1. Side slopes 2:1 - 12" min. mattress thickness.
2. Side slopes 1:1 - 18" min. matress thickness with gabion toe wall if only banks are lined.
3. Side slopes 1:1 - design as a retaining wall using gabions.
4. Channel linings should be tied to the channel banks with gabion counter-forts at least every 21 feet (every 9 feet for higher velocities).

Counterforts should be keyed at least 12 inches into the existing bank.

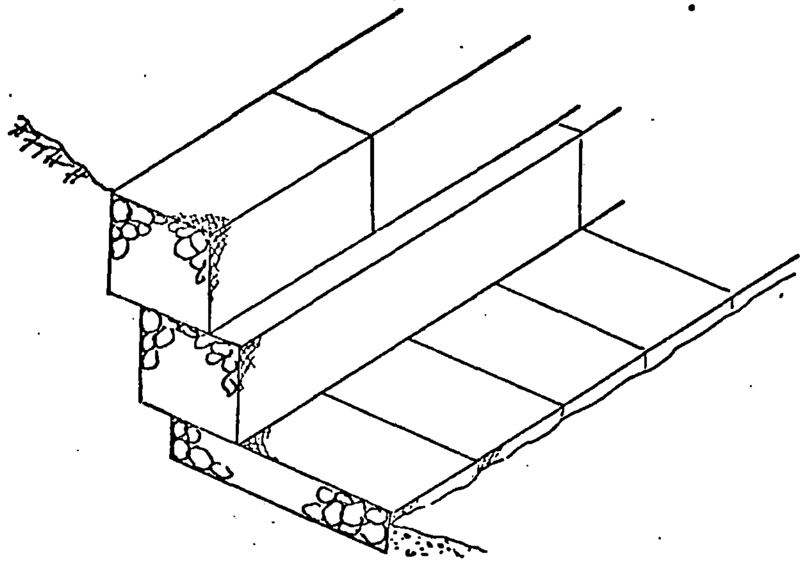
5. Mattresses on channel side slopes need to be tied to the banks by 1/2 inch steel stakes driven 4 feet into tight soil (clay) and 6 feet into loose soil (sand). This is primarily to stabilize the baskets while they are erected and filled with rock. Suggested spacing of the stakes is every six feet along the slope and every eight feet down the slope for slopes 2-1/2:1 and steeper; every 12 feet along the slope as well as down the slope for slopes in excess of 6:1. No stakes are required for slopes of 6:1 and less. Stakes should be placed at basket connection points if possible and should be an integral part of the basket wall, not simply tied to the basket with wire.



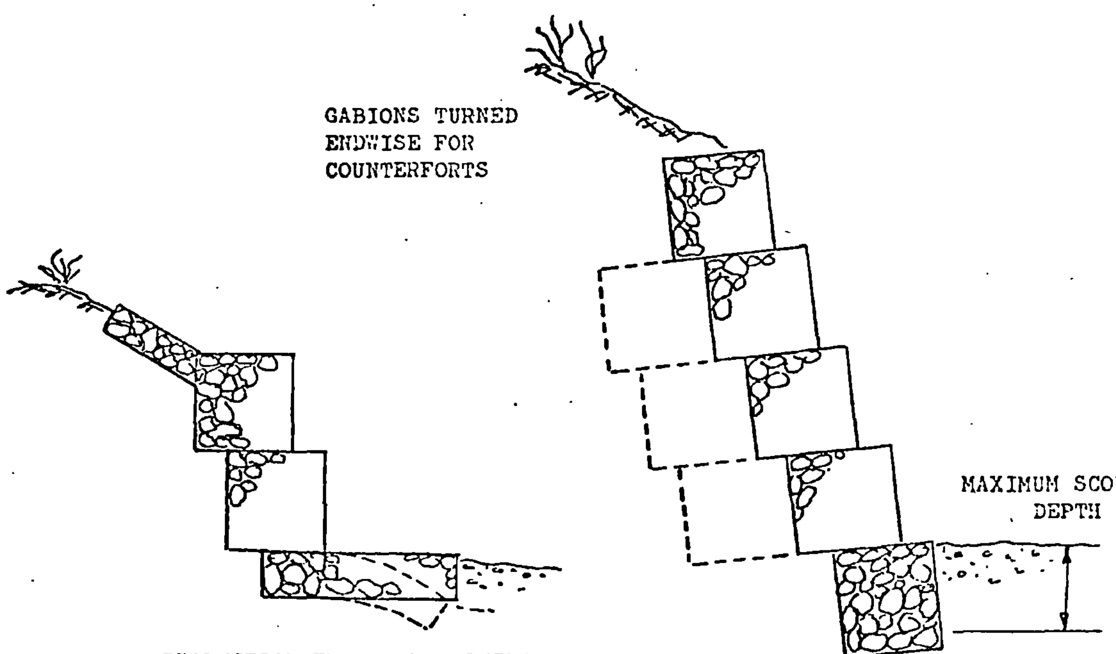
REVETMENT MATTRESS LINING



REVETMENT MATTRESS LINING CONFIGURATIONS



GABION LINING - DESIGN AS A RETAINING WALL



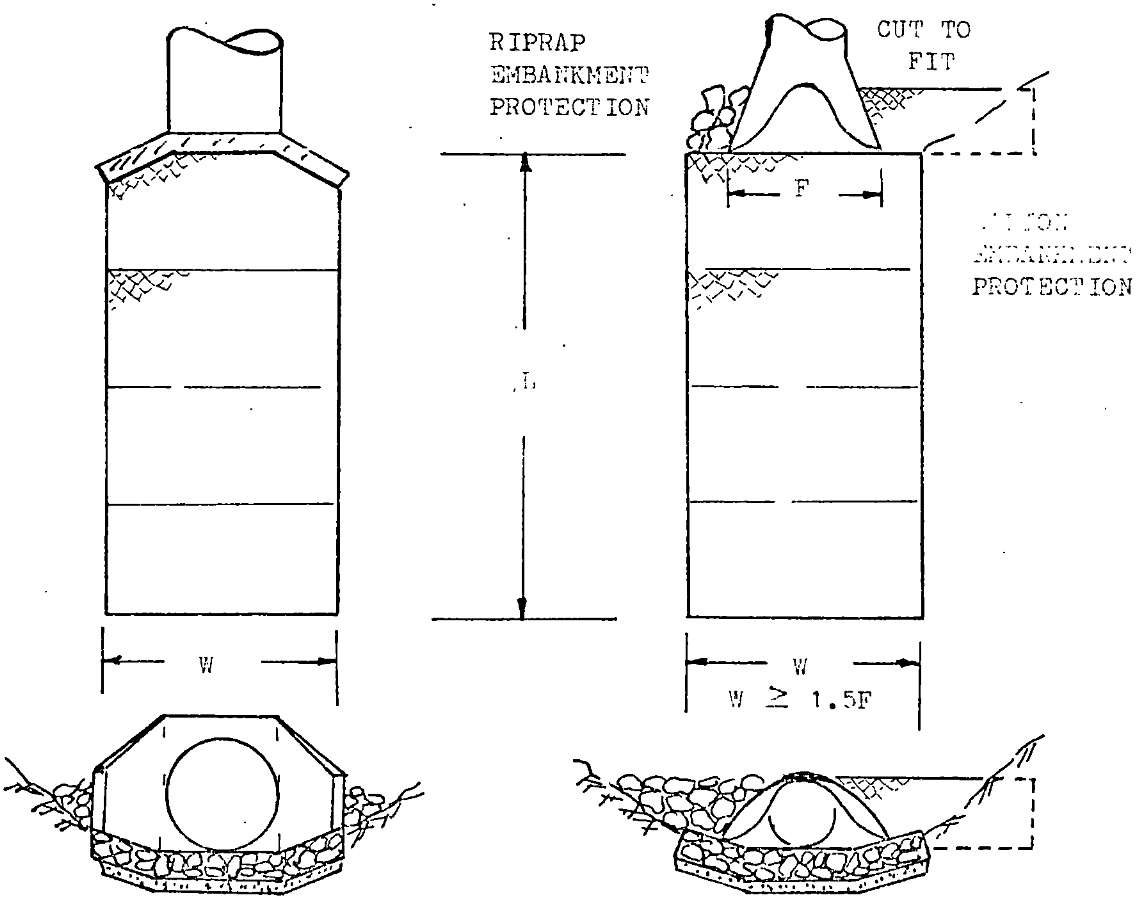
GABION LINING CONFIGURATIONS

B.) Culvert outlet aprons

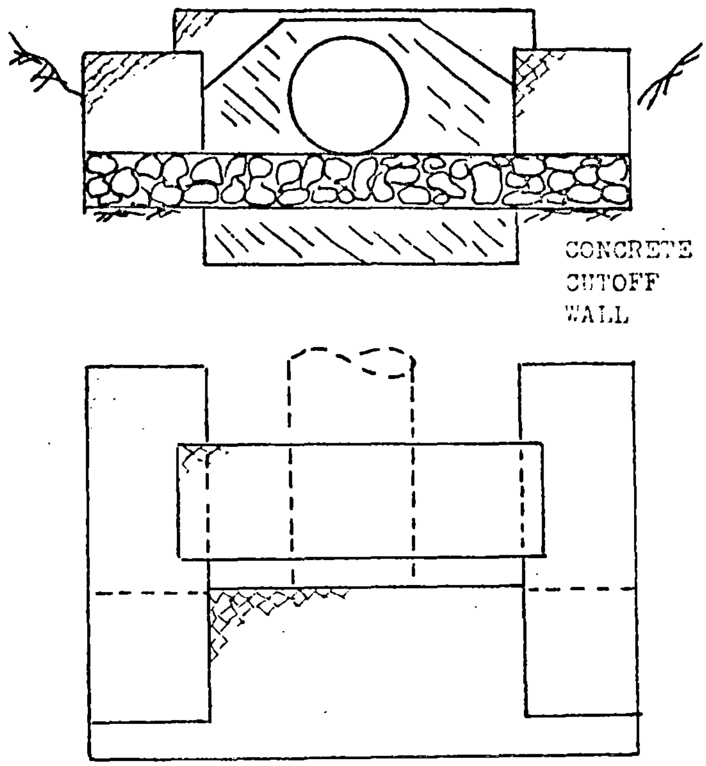
1. Outlet velocity 7-10 fps - 12" min. mattress thickness extending at least 9 ft beyond outlet.

2. Outlet velocity 10-15 fps - 18" min. mattress thickness extending at least 12 to 22 feet beyond outlet.

3. Outlet velocity > 15 fps - energy dissipator required.



CULVERT APRON - SCOUR PROTECTION



CULVERT EMBANKMENT PROTECTION

C.) Weirs, checkdams and drop structures

1. Spacing $= \frac{100h}{s_1 - s_2}$

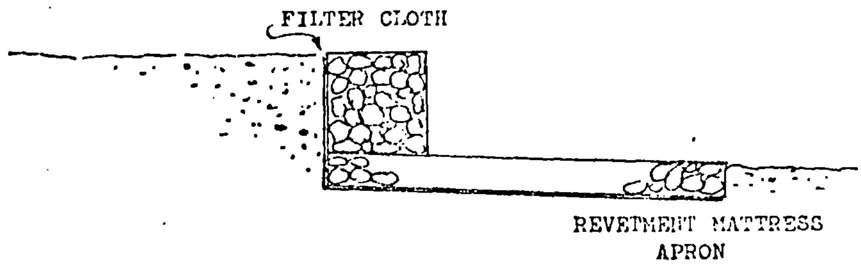
   s_1 = original channel gradient-percent

   s_2 = desired channel gradient-percent

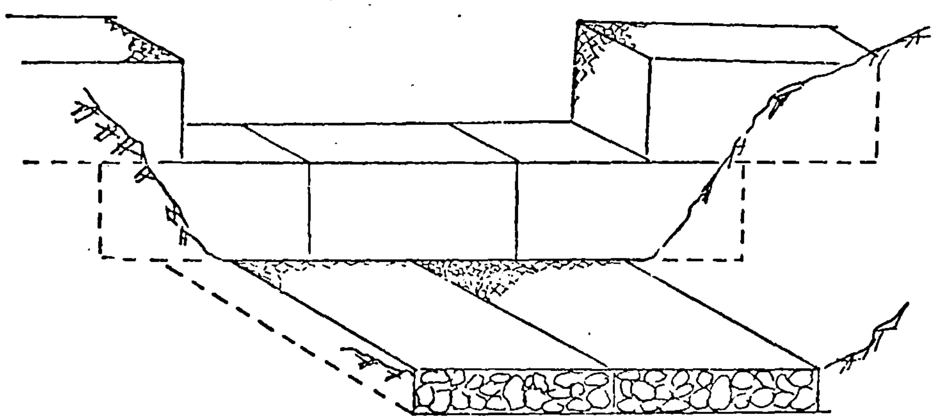
   h = drop -- generally 3 feet maximum

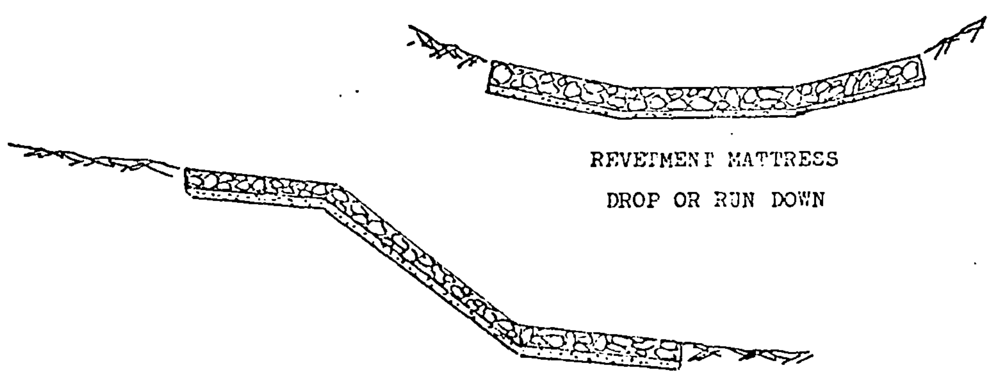
2. Filter or permeable membrane (filter cloth) should be installed on the upstream face of check dams and drop structures to prevent fines from washing through the structure.

3. Downstream aprons are optional depending on the erodability of the bed materials, however either an apron extending 10 ft beyond the structure or a cutoff wall to the depth of expected downstream scour should be provided.



GABION CHECK DAM
OR DROP STRUCTURE

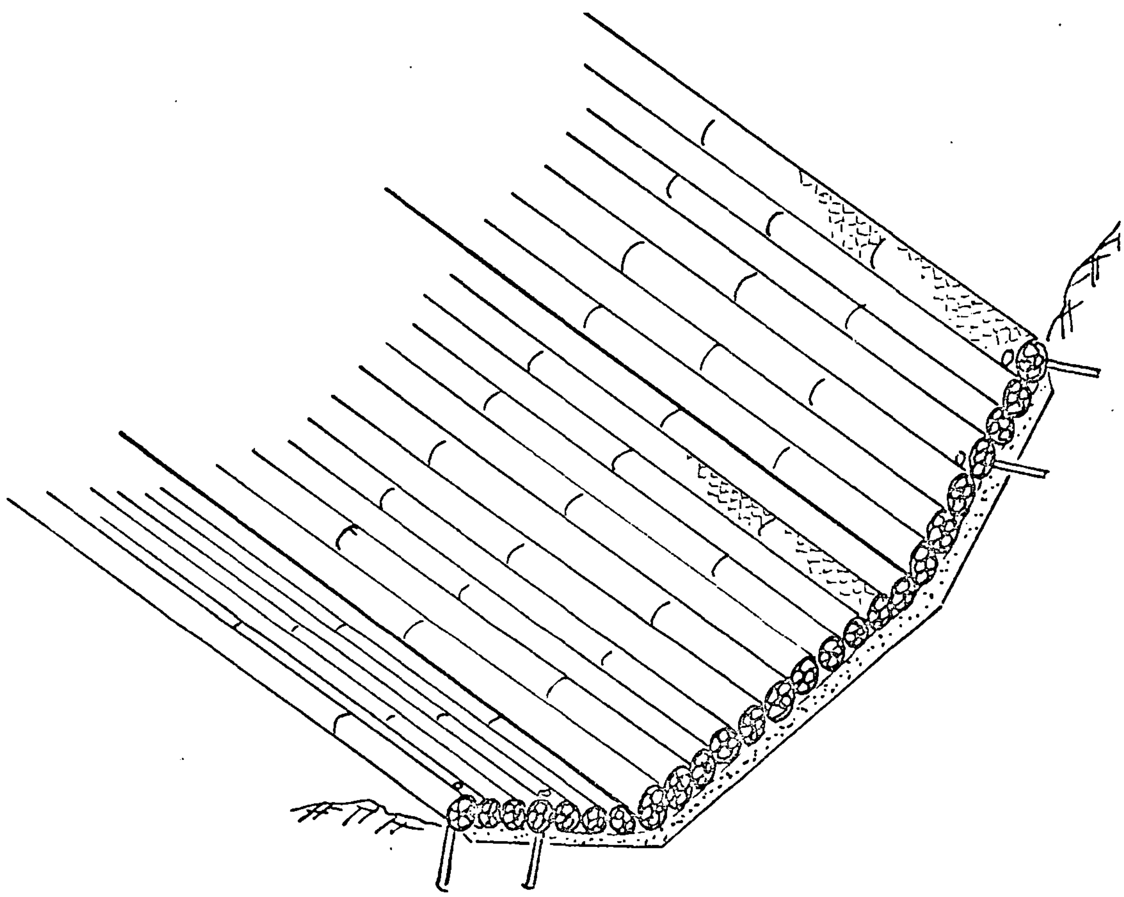




REVETMENT MATTRESS
DROP OR RUN DOWN

D.) Cylindrical wirebound rock

Posey has found through model tests that the diameter of a wirebound rock cylinder need be only about one-half that of loose stones to provide an equivalent resistance to movement (27). As with the rectangular wirebound rock he found that the cylindrical wirebound rock must have sufficient weight to resist scour and must be founded on an adequate filter. Presumably the design rules for rectangular wire encased rock are appropriate for cylindrical wirebound rock as well.



CYLINDRICAL WIRE BOUND ROCK CHANNEL LINING

Table VI-1

Unit Weight of Fill Material

Fill Material	lb/ft^3	lb/yd^3
Basalt	103	2781
Brick	78	2106
Broken Concrete	84	2268
Granite	100	2700
Limestone	90	2430
Sandstone	87	2350
Shingle	94	2538
Slag	94	2538

*Mecafferi Handbook

EXAMPLE VI-1:

Design a wire encased rock apron for a 48" culvert carrying 110 cfs. The tailwater depth is one foot with a velocity of 5 fps, and flow is subcritical.

$$\frac{Q}{D^{5/2}} = \frac{110}{(4)^{5/2}} = 3.44 \,,$$

$$\frac{d_t}{D} = \frac{1}{4} = .25.$$

From Figure V-6

$$\frac{Y_o}{D} = 0.6 \text{ and } Y_o = 2.4'.$$

For $\frac{Y_o}{D} = .6$ from the hydraulic characteristics of a circular section

$$\frac{A_o}{A} = .626 \ ,$$

so that flow area at the culvert exit

$$A_o = \frac{\pi(4)^2}{4} \times .626 = 7.88 \text{ ft}^2$$

$$V_o = \frac{110}{7.88} \cong 14 \text{ fps.}$$

From the suggested design rules this would indicate a minimum revetment mattress thickness of 18" and a minimum apron length of 20 to 22 feet, since the velocity is high.

From Figure V-13, the tangent of the required flare angle is 0.25, so that the downstream width of the apron is:

$$W = D + 2L \tan \theta = 4 + 2L(.25)$$

At this downstream location the velocity is 5 fps at a depth of one foot.

Since Q = AV,

$$110 \text{ cfs} = [4 + 2L(.25)] \text{ ft}(1 \text{ ft})(5 \text{ fps}).$$

Apron length, L = 36 feet which exceeds 22 feet.

Apron width, W = 4 + 2(36)(.25) = 22 feet.

Assume limestone fill with an average size, $K_m = 6$ inches. The channel velocity just downstream from the pipe = 14 fps at depth of 2.4 feet so that

$\frac{K_m}{Y} = \frac{0.5}{2.4} = 0.2$. From Figure VI-1,

$$\frac{V_s}{V_c} = 0.6 \ ,$$

and the Velocity against the stone, $V_s = 0.6 \times 14 = 8.4$ fps. Again, from Figure VI-1, the required weight for a stable isolated cube would be 420 lb.

A rectangular basket, 1.5 x 3 x 12 has a volume of 54 ft^3 and from Table VI-1, limestone filler has a unit weight of 90 lb/ft^3, so that the basket weighs about 4,800 lb -- well in excess of the required 420 lb.

The volume of a cylindrical basket would be $\frac{d}{4} \times L$. Since cylindrically wound wire is available in lengths to 12 feet, assume L = 12' then the volume would be 9.42 d^2. Since, for stability $\frac{90 \text{ lb}}{ft^3} \times 9.42 \, d^2 = 420$ lb, the minimum diameter of the cylinder, d, would be 0.703 feet. Note that this is approximately half the effective riprap size for the non-scour basin from Example V-1, which seems to verify Posey's design rule for "rock sausages".

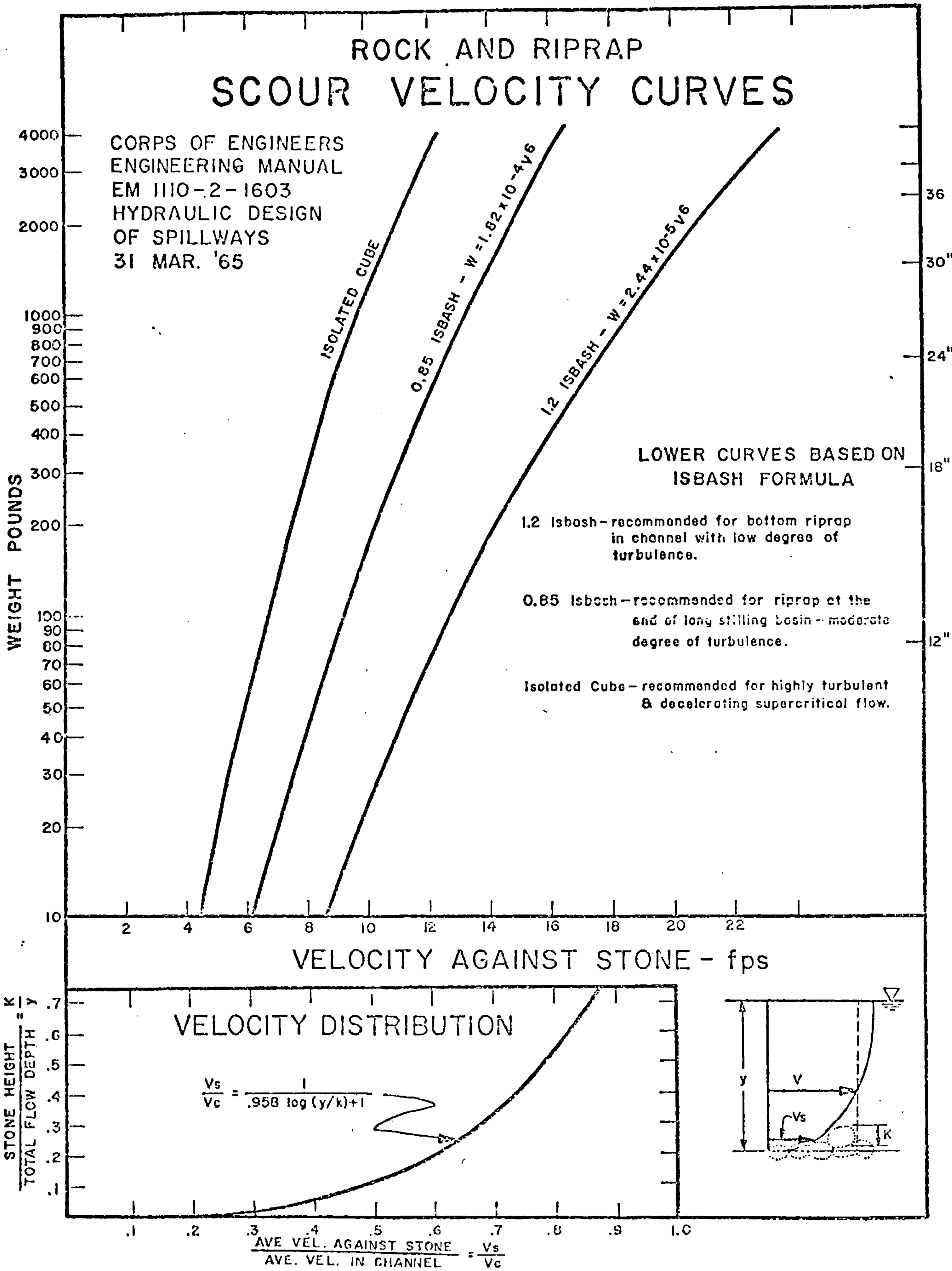

ROCK AND RIPRAP
SCOUR VELOCITY CURVES
CORPS OF ENGINEERS
ENGINEERING MANUAL
EM 1110-2-1603
HYDRAULIC DESIGN
OF SPILLWAYS
31 MAR. '65
ISOLATED CUBE
0.85 ISBASH - W = 1.82 x 10-4 V6
1.2 ISBASH - W = 2.44 x 10-5 V6
LOWER CURVES BASED ON ISBASH FORMULA
1.2 Isbash - recommended for bottom riprap in channel with low degree of turbulence.
0.85 Isbash - recommended for riprap at the end of long stilling basin - moderate degree of turbulence.
Isolated Cube - recommended for highly turbulent & decelerating supercritical flow.
WEIGHT POUNDS
4000
3000
2000
1000
900
800
700
600
500
400
300
200
100
90
80
70
60
50
40
30
20
10
36
30"
24"
18"
12"
2
4
6
8
10
12
14
16
18
20
22
VELOCITY AGAINST STONE - fps
VELOCITY DISTRIBUTION
Vs/Vc = 1/(.958 log (y/k)+1)
STONE HEIGHT / TOTAL FLOW DEPTH = K/y
.7
.6
.5
.4
.3
.2
.1
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
AVE VEL. AGAINST STONE / AVE. VEL. IN CHANNEL = Vs/Vc
y
V
Vs
K

Fig. VI-1

VII - HYDRAULIC JUMP AND IMPACT STILLING BASINS

Both the hydraulic jump stilling basin and the impact stilling basin are well known terminal structures for relatively large spillways and conduits. The complex flow conditions in these basins are not amenable to theoretical analysis and design procedures must be developed empirically. Several standard basins and associated design criteria have been established from model studies for large concrete structures, and will not be duplicated here since these design methods are readily available in the literature. The purpose of this chapter is to provide design criteria for small spillway and conduit terminal structures constructed of materials other than concrete.

Hydraulic Jump Basins

The hydraulic jump can be an efficient energy dissipator when the flow condition entering the basin approximates uniform supercritical flow with a Froude number in excess of two, and the downstream tailwater is of sufficient depth to produce a stable hydraulic jump over the range of expected discharges. These conditions exist at the transition from a steep sloping channel to a mild sloping channel such as the termination of a spillway, and may exist at the outlet from a conduit with the control at the inlet. There are several

standard hydraulic jump basins described in the literature which will not be duplicated here (11). These basins are constructed of concrete and the design is based on model studies using smooth model surfaces. The characteristics of the hydraulic jump are somewhat different over a rough surface, such as wire encased rock, particularly the length of the hydraulic jump which may be significantly shorter. The purpose of this section is to provide information on hydraulic jump basins which is not readily available in hydraulics texts and manuals, such as the length of a hydraulic jump in trapezoidal channel and the length of a hydraulic jump in a rough channel. It is recommended that the information in this section be used to design stilling basins only for small structures.

The sequent subcritical depth associated with given supercritical flow conditions may be estimated from design charts found in most hydraulics texts, or manuals for rectangular channels, or computed by applying the momentum principle for any shaped channel. A common form of the momentum, or specific force equation is:

$$\beta_1 \frac{Q^2}{gA_1} + \bar{Z}_1 A_1 = \beta_2 \frac{Q^2}{gA_2} + \bar{Z}_2 A_2 = \text{constant} \qquad \text{(VIII-1)}$$

where

A = flow area

$\bar{Z}$ = distance from the water surface to the centroid of the flow area

β = coefficient which compensates for non-uniform velocity distribution.

Equation (VII-1) requires an additional correction factor when the flow eminates from a conduit with an M-2 or S-2 water surface profile, since a non-hydrostatic pressure distribution occurs in the conduit. The momentum, or specific force at a conduit outlet is given by

$$\beta \frac{Q^2}{gA} + \beta'\bar{Z}A = \text{constant} \qquad \text{(VII-2)}$$

where β' compensates for the non-hydrostatic pressure distribution. For rectangular conduits $\beta' = 0.65$ is recommended for design purposes, and unless better information is available β can be assumed equal to one for all channels (40).

Figure VII-2 provides a quick means to determine the sequent depth for horizontal channels. Figure VII-1 provides a method for estimating the length of a hydraulic jump in a smooth trapezoidal channel and Figure VII-3 is based on the model studies performed by Nass, at the University of Colorado, and provides a means for estimating the length of a hydraulic jump in a rectangular wire encased rock channel. The use of the latter will be demonstrated in the following example.

EXAMPLE VII-1

A 4 foot by 6 foot rectangular concrete conduit on a slope of 3 percent enters a trapezoidal earth channel with 2:1 side slopes, n = 0.03, four foot base width and slope of 0.1 percent. The maximum discharge is 200 cfs and the length of the conduit is 200 feet. Design a 4 foot wide rectangular hydraulic jump stilling basin made of wire encased rock. Assume normal depth and n = 0.015 for the conduit.

The basin must operate for all discharges equal to and less than the maximum discharge, therefore, the analysis will be made using discharges of 200 cfs, 100 cfs and 50 cfs. The tailwater depth is assumed to be the normal depth in the downstream channel.

Q	Tailwater depth
200	4.9 ft
100	3.6 ft
50	2.6 ft

Assuming uniform supercritical flow in the conduit, the flow characteristics for the three assumed discharges would be as shown in the table below.

Rectangular Conduit @ 3% Slope

Q cfs	Normal Depth ft.	V fps	Froude No.	Sequent Depth ft.
200	2.6	19.2	2.09	6.5
100	1.6	15.6	2.17	4.2
50	1.0	12.5	2.20	2.7

Sequent depth is computed by trial from the momentum equation (eq. VII-1). Calculations assume that a hydrostatic pressure distribution exists in the conduit ($\beta' = 1$) since the conduit acts as a steep open channel rather than a pipe.

Note that the sequent depths required to produce a stable hydraulic jump exceed the available tailwater depths for all trial discharges. Therefore the basin must be at least 1.6 (i.e., 6.5 - 4.9) feet lower than the channel bed to effectively raise the tailwater depths so that they equal or exceed the required sequent depths. If the conduit slope is increased to 4 percent the conduit exit will be two feet lower than with a 3 percent slope. Therefore increase the conduit slope to 4 percent.

Rectangular Conduit @ 4% Slope

Q cfs	Normal Depth ft.	V fps	Froude No.	Sequent Depth ft.	Sequent Depth less 2 feet
200	2.40	20.8	2.37	6.9	4.9
100	1.40	17.8	2.65	4.7	2.7
50	0.87	14.4	2.72	2.9	0.9

Comparison of the last column of the above table with the appropriate tailwater depths indicates that the hydraulic jump will be contained on the basin, particularly since the sill formed by lowering the basin in relation to the channel bed has an added stabilizing effect. At lower discharges the hydraulic jump will be drowned out at the conduit exit.

From Figure VII-1, the length of a hydraulic jump on a smooth horizontal surface ($K \rightarrow \infty$) is given by the relationship $\frac{L_s}{y_1 - y_2} = 6.9$. Therefore, $L_s = (6.9)(6.9 - 2.4) = 31$ feet at 200 cfs. From figure VII-3 the length of the same hydraulic jump on a horizontal wire encased rock basin would be $0.8 \times 31 = 24.8$, say 25 feet. Note that at higher Froude numbers a rough basin produces a hydraulic jump of only about one half the length of a similar jump over a smooth surface.

Impact Basins - Impact basins are particularly advantageous when the downstream tailwater conditions are not sufficiently high to maintain a hydraulic jump in the basin and provides about the only means for dissipating kinetic energy in steeply sloped channels. An impact basin, as the name implies, dissipates kinetic energy through the impact of the high velocity jet on a rigid baffle. Since a significant force is imparted to the baffle, the impact basin must be securely anchored to the channel banks and bottom. The magnitude of the thrust imparted to the basin can be estimated from the momentum equation in the form:

$$\Sigma\vec{F} = \rho Q(\vec{V}_2 - \vec{V}_1) \qquad \text{(VII-3)}$$

Three separate basins (although two are not strictly impact basins) will be described in the following pages.

A. Forest Service Impact Energy Dissipator - The Forest Service Impact Energy Dissipator was developed for the Forest Service from model studies performed by the Bureau of Reclamation (8). The light weight sheet metal basin was developed for circular conduits from 18 to 36 inches in diameter, and can be fabricated in a shop to be carried to the field for assembly. The model studies showed that for flow conditions within 15 percent of the design discharge the basin exhibited excellent self cleaning characteristics, while at lower flow rates the debris cleaning potential was significantly

reduced. The model studies indicated that unless a back water condition was produced by the downstream channel, flow will be at critical depth across the exit lip. Riprap or wire encased rock should be provided for erosion protection immediately downstream from the exit lip. In addition, at conduit slopes greater than 40 percent, corner fillers are (see figure VII-6) required to maintain proper flow characteristics. The critical dimensions are the baffle and deflector location, upstream dissipator width, downstream wall divergence and the exit lip elevation. The dissipator dimensions are based on the design discharge and conduit slope. The dimension multiplier, F_I, for use with figures VII-5 and VII-6 is computed with equation VII-3 and Table VII-1. The design procedure was developed from model studies for an 18 inch diameter conduit. Adjustments for pipes other than 18 inches in diameter will follow in a subsequent paragraph.

$$F_{I_{18}} = C\left[Q_{18}\right]^{0.4} \qquad \text{(VII-4)}$$

Table VII-1

Conduit Slope Percent	C	Maximum $F_{I_{18}}$
5	0.280	0.83
10	0.280	0.90
20	0.285	1.02
30	0.290	1.13
40	0.310	1.29
50	0.330	1.40
60	0.350	1.40

Note that $F_{I_{18}}$ has a maximum value which corresponds to full pipe flow. If $F_{I_{18}}$ exceeds the maximum, another type of energy dissipator should be selected. The design procedure was developed for an 18 inch diameter conduit and for pipes other than 18 inches in diameter the following adjustment procedure is required:

1. Equivalent design discharge

   $$Q_{18} = Q_D \Big/ \left(\frac{D}{18}\right)^{2.5}$$

   where Q_D = the actual design discharge and D = the pipe diameter in inches.

2. The dimension multiplier, $F_{I_{18}}$, from equation VII-4 must be multiplied by the diameter ratio.

$$F_{I_D} = F_{I_{18}} \left(\frac{D}{18}\right)$$

The vertical position of the conduit invert, Y_B, is determined by the conduit and channel geometry, and the elevation of the exit lip. Once Y_B has been established the horizontal position of the conduit invert, X_B, is determined from figure VII-4. X_B should be at least as large as the conduit diameter and in no case less than 18 inches.

EXAMPLE VII-2

Determine the required deflector location, backplate width, backplate height and exit lip width for a Forest Service Impact Energy Dissipator which is connected to an 18 inch corrugated metal pipe on a 30 percent slope. The design discharge is 15 cfs. Assume no constraints on the pipe location with respect to the exit lip.

Since this is an 18 inch pipe no diameter adjustments will be required.

$$F_I = (0.29)(15)^{0.4} = 0.86$$

From figure VII-5:

Backplate width = 0.86(54) = 46.5 inches

Backplate height = 0.86(72) -0.86(1.8) = 49.5 inches

Exit Lip width = 0.86(72) = 62 inches

Since there are no constraints on the pipe location, use minimum X_B and Y_B values for the deflector location.

X_B = 18 inches (minimum X_B = pipe diameter)

From figure VII-4

Y_B = 14 inches

EXAMPLE VII-3

Determine the dimension multiplier, F_I, required to size a Forest Service Energy Dissipator for a 24 inch pipe carrying a discharge of 30 cfs on a 30 percent slope.

The equivalent design discharge:

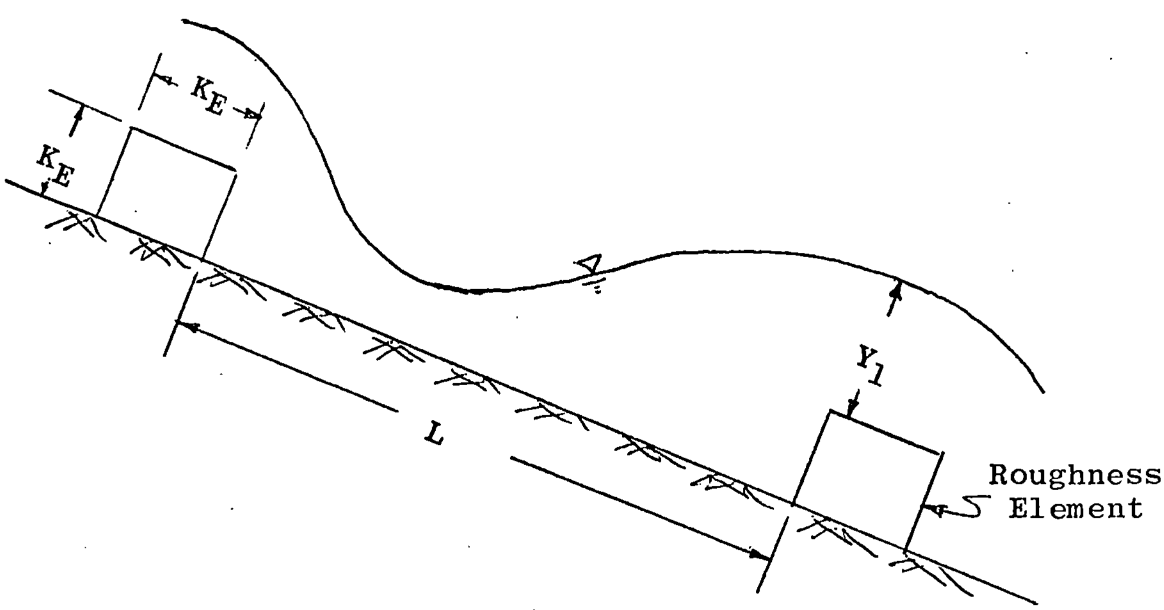
$$Q_{18} = 30 \Big/ \left(\frac{24}{18}\right)^{2.5} = 14.6 \text{ cfs}$$

$$F_{I_{18}} = (0.29)(14.6)^{0.4} = 0.85$$

Note that $F_{I_{18}}$ does not exceed the maximum F_I value in Table VII-1.

$$F_I = 0.85 \left(\frac{24}{18}\right) = 1.13$$

B. Roughness Element Energy Dissipator - The roughness element energy dissipator makes use of square roughness bars placed across the channel bottom perpendicular to the direction of flow to control the high velocities generated in steep chutes and conduits. Kinetic energy is dissipated in a series of small hydraulic jumps within the channel, a condition referred to as tumbling flow. Roughness elements must be properly sized and located so that neither an unstable tumbling condition (pulsating surges) nor rapid flow conditions (water skims over the roughness elements) occur. In the model studies which resulted in the design criteria described below, roughness element energy dissipators were found to limit the exit velocities from steep chutes to slightly less than the channel critical velocity (21)(49).



Tumbling Flow

1. Design criteria for rectangular channels with channel slopes greater than two percent.

a) Roughness element height and width:

$$K_E = \frac{Y_c}{(3 - .0375)^{2/3}}$$

where Y_c is the channel critical depth, $(q^2/g)^{1/3}$, and S is the bed slope in percent. Note Y_c and K_E are in feet.

b) Roughness element spacing:

$$7.5 \leq L/K_E \leq 12.5$$

c) Flow depth over roughness element

$$Y_1 = 0.35(q)^{2/3}$$

d) Exit velocity: (assumed to be critical velocity)

$$V = (gq)^{1/3}$$

e) Drag force on each roughness element:

$$F_D = \frac{1.29\ K_E\ q^2}{(K_E + Y_1)^2}\left[\left(\frac{K_E}{Y_1}\right)^3 + 4\left(\frac{K_E}{Y_1}\right)^2 + 2.2\left(\frac{K_E}{Y_1}\right) + 0.6\right]$$

f) At least five rows of roughness elements must be provided.

2. Design criteria for trapesoidal channels with slopes greater than one percent. The criteria above is applicable to trapezoidal channels

using $q = Q/B$ where B is the channel bottom width.

3. Design criteria for circular conduits with slopes greater than 4 percent.

   a) Roughness elements are annular rings of square cross section set perpendicular to the direction of flow.

   b) Roughness element height and width:

$$0.104 \le K_E/D \le 0.146$$

   c) Roughness element spacing:

$$1.5 \le L/D \le 2.5$$

   d) Required conduit diameter:

$$D = \left[Q^2/0.0625g\right]^{1/5}$$

   e) Exit velocity = critical conduit velocity.

   f) At least five roughness elements in the downstream end of the pipe are required.

EXAMPLE VII-4

An earth channel carrying 200 cfs is roughly trapezoidal with 2 to 1 side slopes, a 4 foot bottom width, $n = 0.03$ and a 4 percent bed slope. The channel passes through a 4 x 6 foot rectangular conduit for 200 feet, then returns to the natural channel. Roughness elements are to be placed in the conduit to limit the exit velocity. Determine the required size and spacing of the

roughness elements, and estimate the drag force on each element. Compare the conduit exit velocity with the velocity of the natural channel.

Critical velocity in the conduit:

$$Y_c = (q/g)^{1/3} = [(50)/g]^{1/3} = 4.26 \text{ ft.}$$

Required roughness element height:

$$K_E = \frac{4.26}{[3 - .037(4)]^{2/3}} = 2 \text{ feet}$$

Since the roughness elements are square the width is also 2 feet.

Roughness element spacing:

$L = 7.5(2) = 15$ ft. lower limit

$L = 12.5(2) = 25$ ft. upper limit

Use 10 roughness elements spaced 20 feet apart, with the last element 10 feet from the pipe exit.

$$Y_1 = 0.35(50)^{2/3} = 4.8 \text{ ft.}$$

Note that the top of the conduit may interfere with the performance of the roughness elements since $Y_1 + K_E = 4.8 + 2 = 6.8$ ft. which is greater than the height of the conduit (6 ft.).

Drag force on each element:

$$F_D = \frac{(1.29)(2)(50)^2}{(6.8)^2}\left[\left(\frac{2}{4.8}\right)^3 + 4\left(\frac{2}{4.8}\right)^2 + 2.2\left(\frac{2}{4.8}\right) + 0.6\right]$$

$$F_D = 318.5 \text{ lb.}$$

Assume the natural channel flows at normal depth:

$$200 = \frac{1.49}{0.03}\left[Y(2Y+4)\right]\left[\frac{Y(2Y+4)}{4.47Y+4}\right]^{2/3} \quad (0.04)^{1/2}$$

Solution by trial yields Y = 2.1 feet and

$$V = \frac{200}{(2.1)(4.2+4)} = 11.6 \text{ fps.}$$

Exit velocity from the conduit:

$$V = \left[(32.2)(50)\right]^{1/3} = 11.7 \text{ fps}$$

so that the exit velocity is approximately equal to the natural channel velocity.

Note from example VII-1 that without the roughness elements the exit velocity would be 20.8 fps.

Weep holes or notches should be provided in the roughness elements, staggered transversely in adjacent elements, to provide drainage at low flows.

C. South Dakota State University Impact Basin - This basin uses a precast concrete or metal sill to dissipate kinetic energy. The basin was developed for use with conduits up to 36 inches in diameter, from model studies preformed by Chang and Karim at South Dakota

State University (7). The basin configuration is shown in figure VII-7 and the design criteria are as follows:

1.) The elevation of the sill top should be level with the pipe invert when discharging onto open ground and (3/4)D above the pipe invert when discharging into a channel.

2.) The sill should be imbedded in the channel banks and bottom sufficiently to withstand the resulting impact forces and to preclude underminning due to scour. The sill should be imbedded no less than (3/4)D into the channel bottom (measured at the center line) and at least (1/2)D into each bank (measured at the top of the sill). When discharging onto open ground the triangular basin should be carried at least one pipe diameter beyond the sill.

3.) The riprap in the triangular basin should be mixed rock sizes ranging from (1/8)D to (3/8)D. Protection is required for the intensive scour just downstream from the sill.

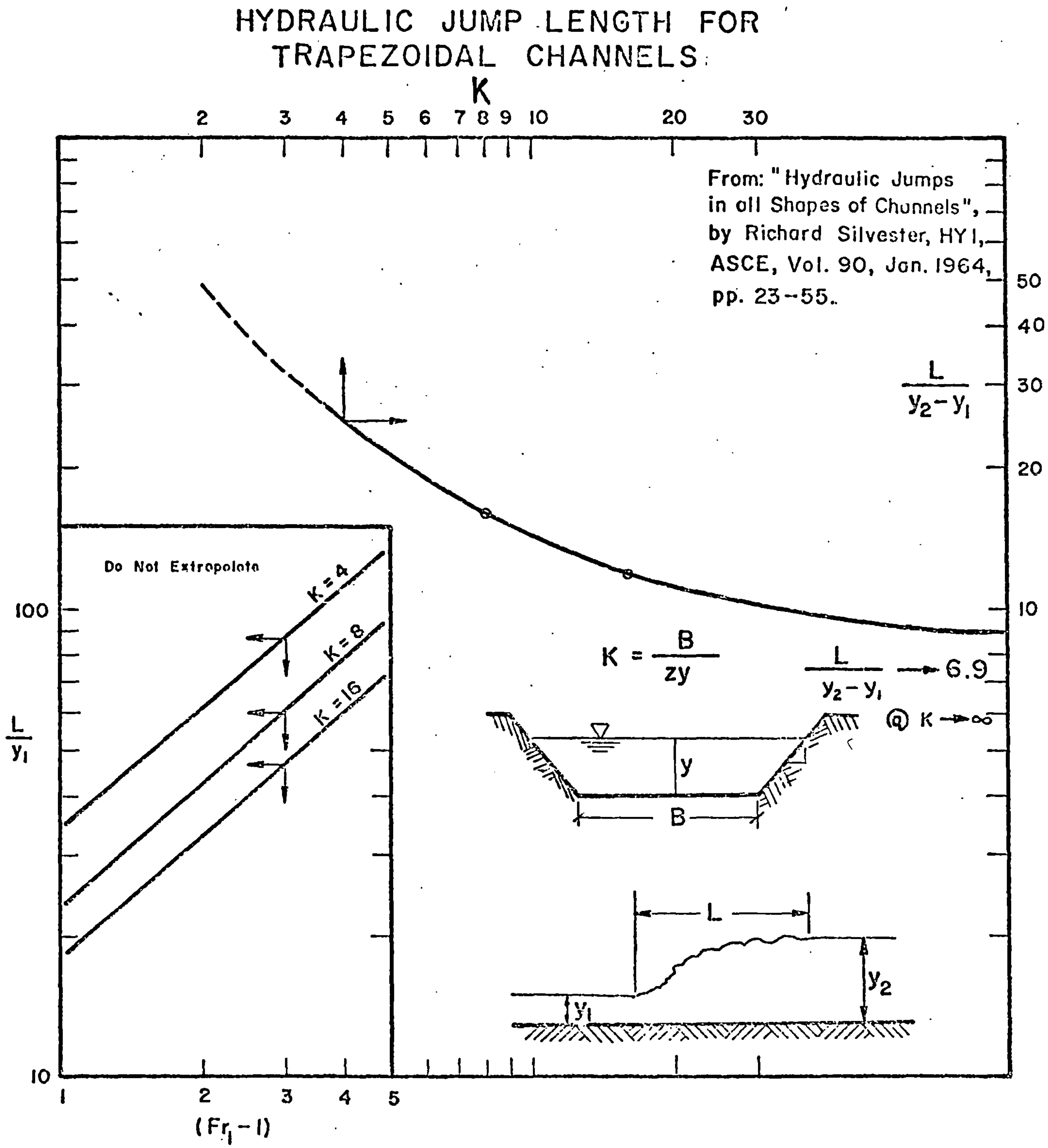

HYDRAULIC JUMP LENGTH FOR TRAPEZOIDAL CHANNELS
K
2 3 4 5 6 7 8 9 10 20 30
From: "Hydraulic Jumps in all Shapes of Chunnels", by Richard Silvester, HY1, ASCE, Vol. 90, Jan. 1964, pp. 23-55.
50
40
30
20
10
L / (y2 - y1)
Do Not Extrapolate
K = 4
K = 8
K = 16
100
10
L / y1
K = B / zy
L / (y2 - y1) → 6.9 @ K → ∞
y
B
L
y2
y1
1 2 3 4 5
(Fr1 - 1)

Fig. VII-1

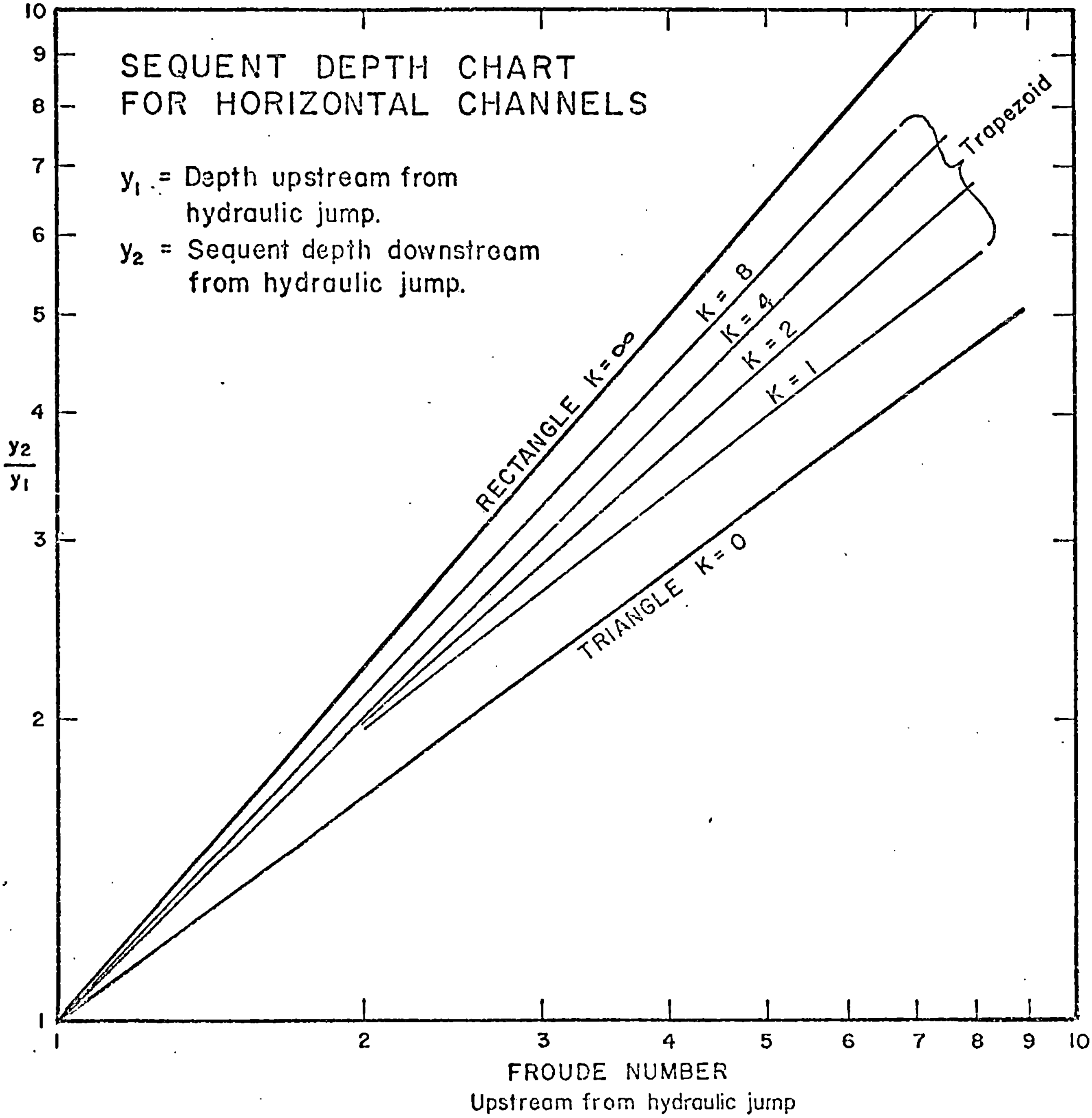

SEQUENT DEPTH CHART
FOR HORIZONTAL CHANNELS
y_1 = Depth upstream from hydraulic jump.
y_2 = Sequent depth downstream from hydraulic jump.
RECTANGLE K=∞
K = 8
K = 4
K = 2
K = 1
TRIANGLE K=0
Trapezoid
$\frac{y_2}{y_1}$
1
2
3
4
5
6
7
8
9
10
FROUDE NUMBER
Upstream from hydraulic jump

Fig. VII-2

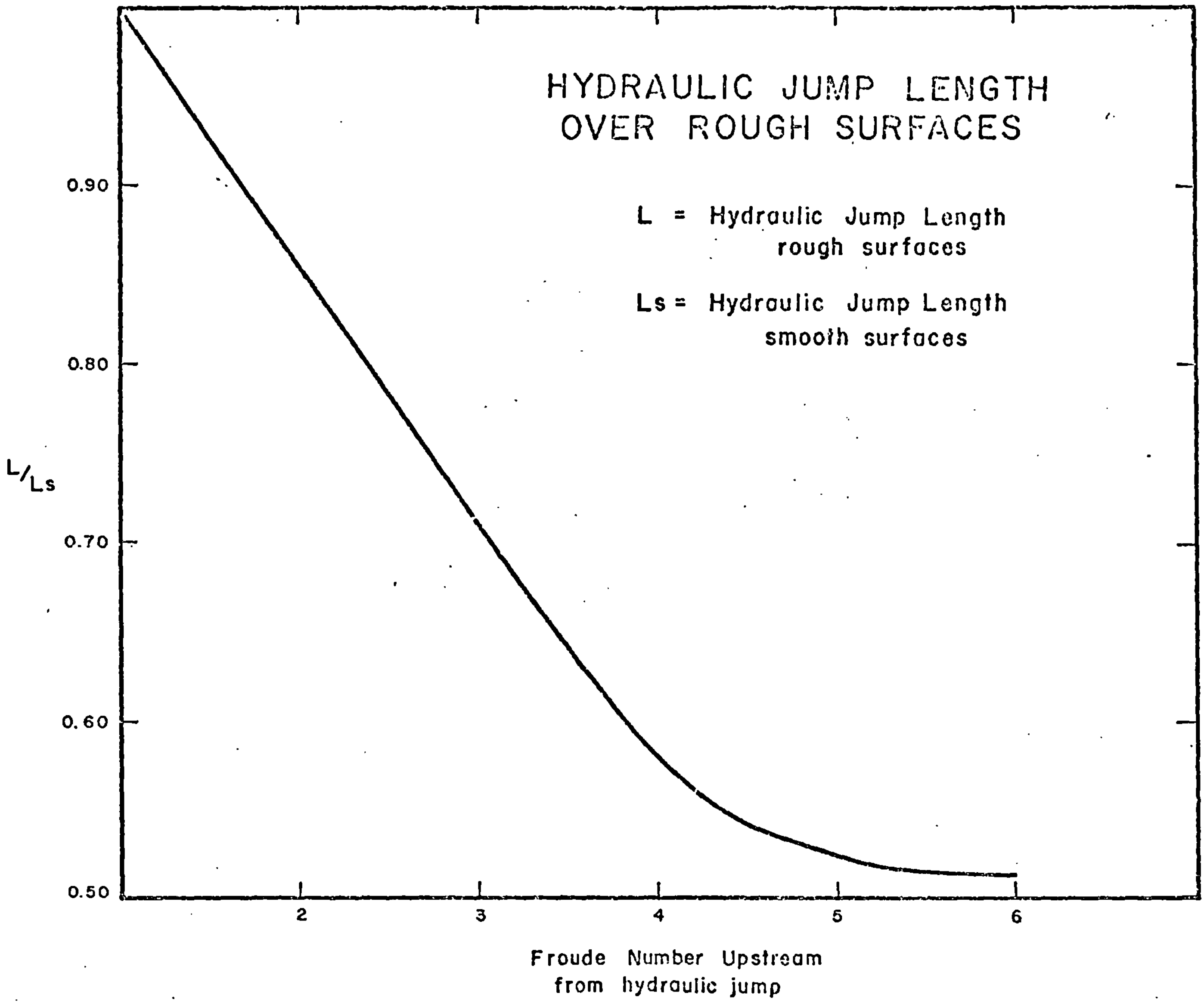
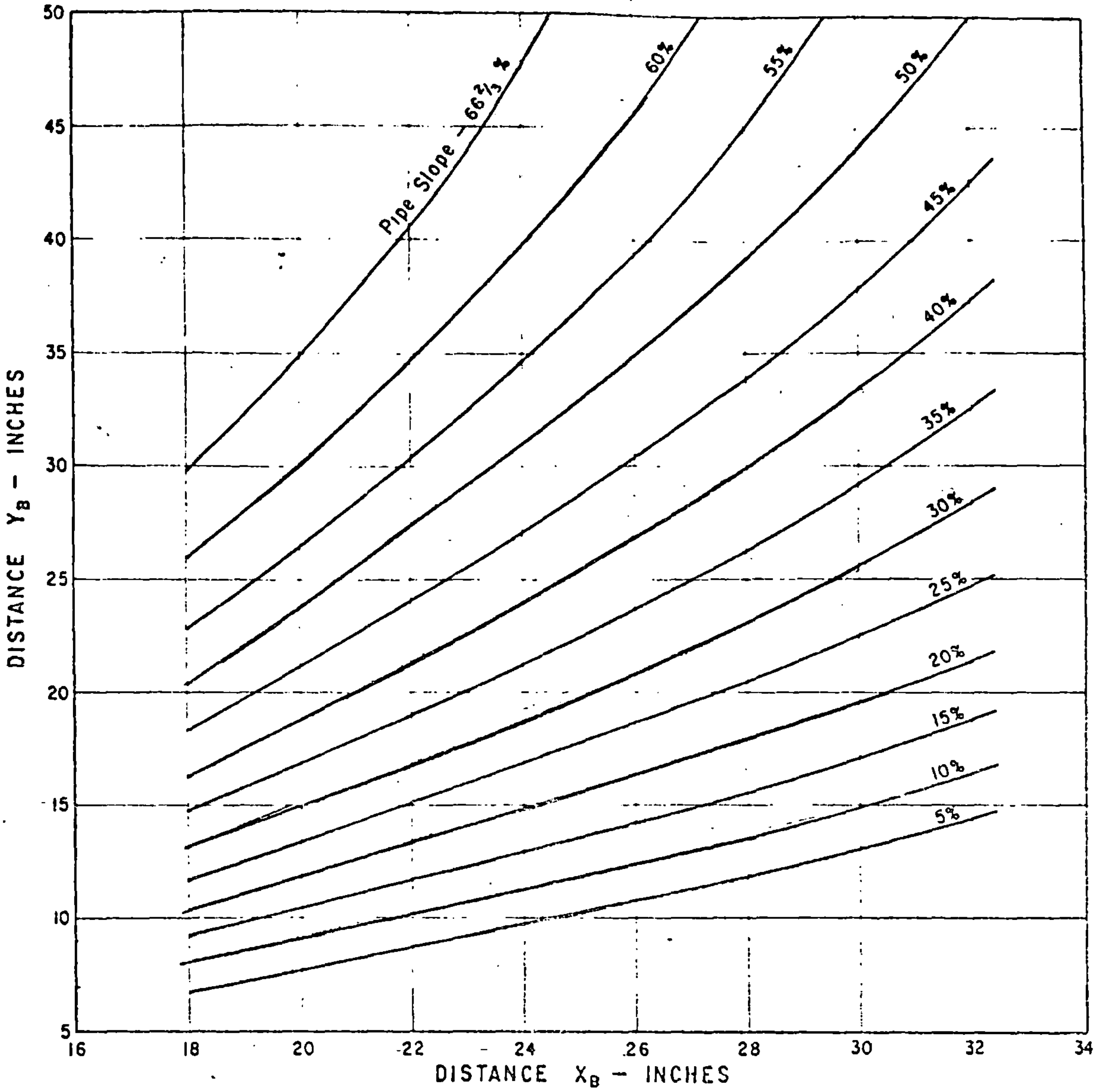
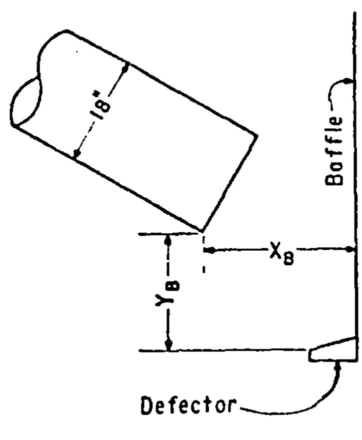

HYDRAULIC JUMP LENGTH
OVER ROUGH SURFACES
L = Hydraulic Jump Length
rough surfaces
Ls = Hydraulic Jump Length
smooth surfaces
0.90
0.80
0.70
0.60
0.50
L/Ls
2
3
4
5
6
Froude Number Upstream
from hydraulic jump

Fig. VII-3



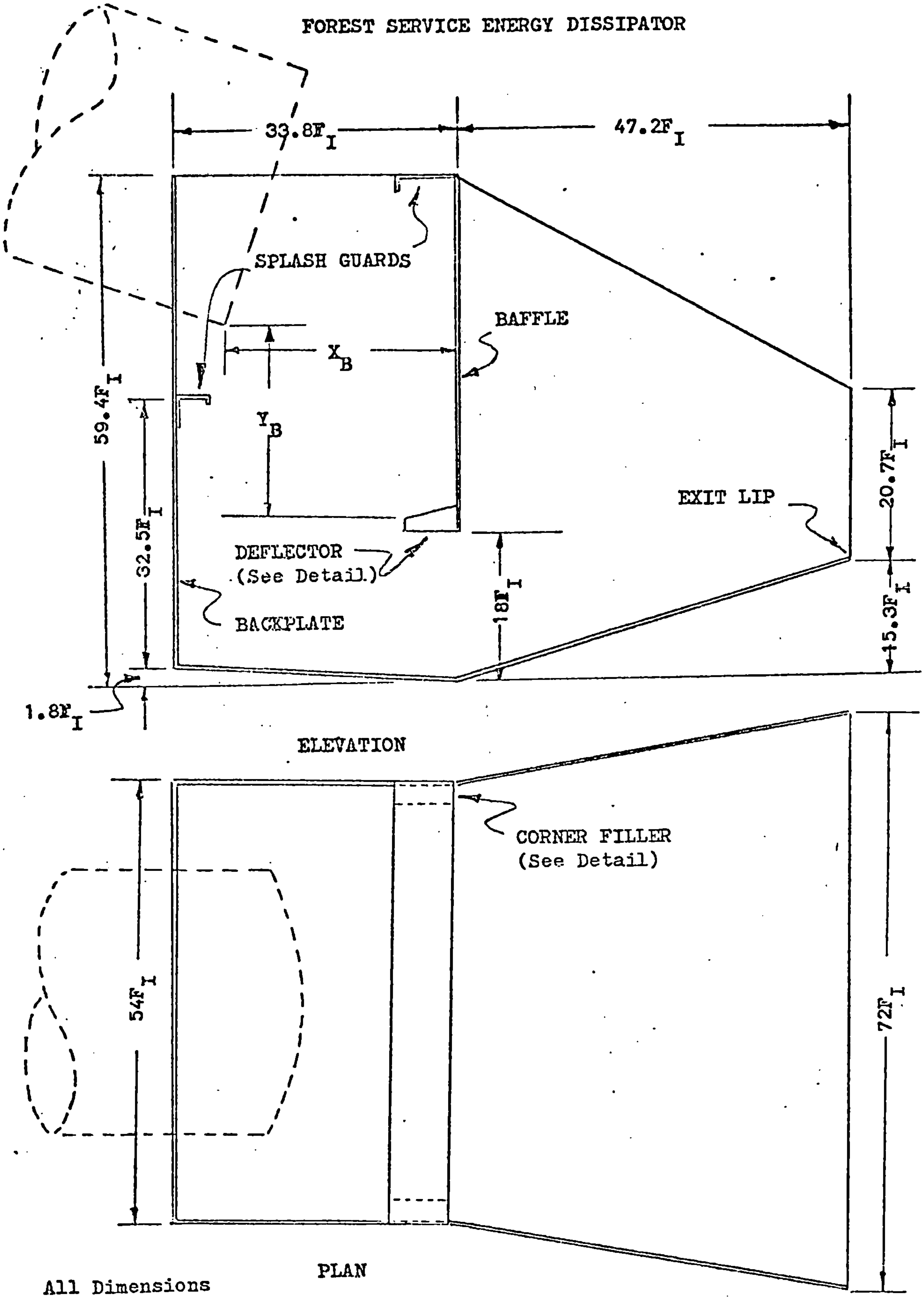


FOREST SERVICE
ENERGY DISSIPATOR
FOR
18-INCH CORRUGATED PIPE

From Reference 8

Fig. VII-4

FOREST SERVICE ENERGY DISSIPATOR



All Dimensions
in inches
From Reference 8

Fig. VII-5

FOREST SERVICE ENERGY DISSIPATOR
DETAIL SHEET

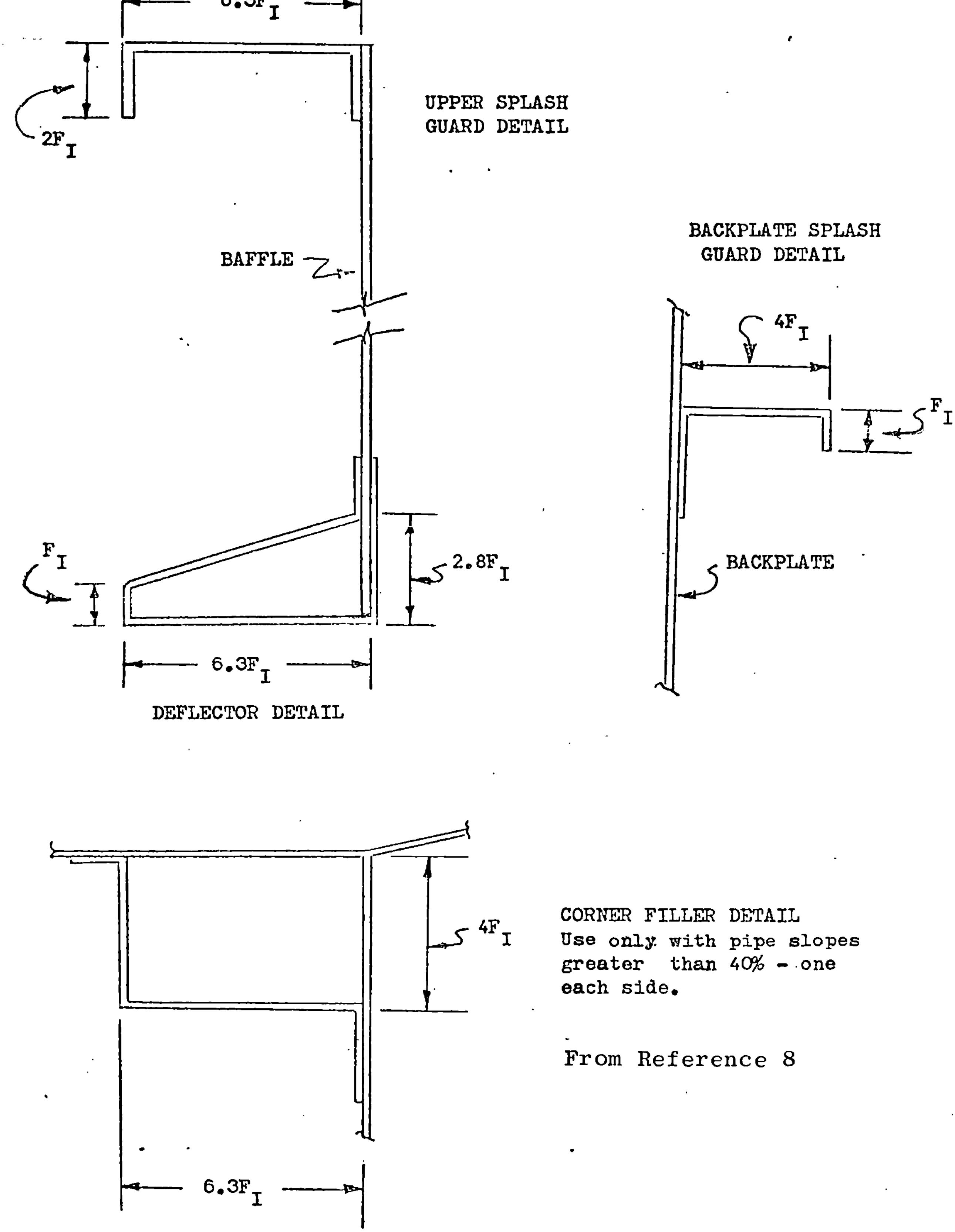
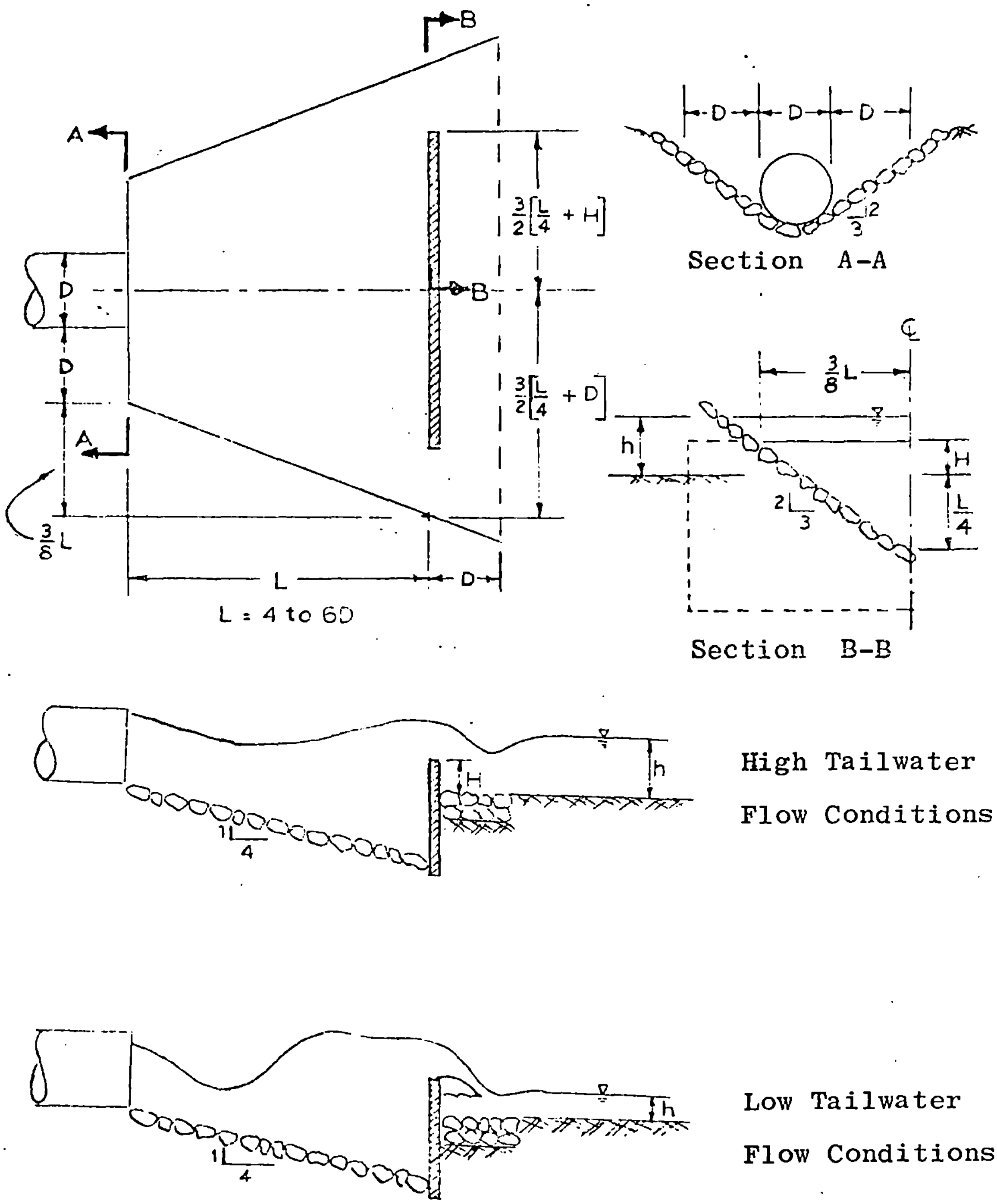


Fig. VII-6

SOUTH DAKOTA STATE UNIVERSITY IMPACT BASIN



From Reference 7

Fig. VII-7

APPENDIX A

RIPRAP SAFETY FACTOR EQUATIONS (54)

$$S.F. = \frac{\text{Moments resisting particle rotation out of bank}}{\text{Moments tending to rotate particle out of position}}$$

The particle submerged unit weight is acted upon by both lift and drag forces. Drag acts parallel to velocity vector and lift acts perpendicular to the bank. Failure results from the stone rolling out of position and downslope.

$$S.F. = \frac{\cos\theta' \tan\Phi}{\eta' \tan\Phi + \sin\theta\cos\beta}$$

$$\beta = \tan^{-1}\left[\frac{\cos\lambda}{\frac{2\cos\theta'}{\eta\tan\Phi} + \sin\lambda}\right]$$

$$\eta = \frac{21\,\tau_s}{(S_s-1)\gamma K} \simeq \frac{21\,RS}{(S_s-1)K} \quad .$$

The above equation requires steady uniform flow, but can be used for accelerating flow. It should not be used where flow is decelerating or below energy dissipating structures.

U_r = velocity at the channel bottom

$U_r = 8.5U_* = 8.5\sqrt{gRS}$

$\eta = \dfrac{0.3\,U_r^2}{(S_s-1)gK}$

U = depth averaged velocity $\approx$ V (mean velocity) in wide channel where R = y

$$\eta = \frac{\epsilon\, U^2}{(S_s - 1) g K}$$

$$\epsilon = .3\left[\frac{3.4}{\ln(12.3\ Y/K)}\right]^2$$

$$\eta' = \eta \left[\frac{1 + \sin\ (\lambda + \beta)}{2}\right]$$

S = factor of safety

θ' = side slope angle

Φ = angle of repose for riprap

β = angle defining direction of particle movement

λ = downslope angle of velocity vector

η = stability number for flow on plane flat bed

η' = stability number for flow on side slope

τ_s = average tractive force

S_s = specific gravity of rock

γ = unit weight water

K = average stone size

S.F. = $\frac{1}{\eta}$ on flat bed or on channel bottom

APPENDIX B

SCOUR PROTECTION AT CONDUIT OUTLETS - CSU CURVES (40)

Appendix B contains design curves from the report, "Flood Protection at Culvert Outlets" by Simons, Stevens and Watts, which were the basis for the design curves for ripraped energy dissipators in Chapter V. The general design procedure is the same as shown in example problems V-1 through V-4, however there are a number of minor differences which need to be explained.

The effective rock size is represented by the symbol d_m rather than K_m as in Chapter V. All other symbols are the same. In example V-1, Table V-3 is developed using arbitrarily selected values of d_s/D (Column (1), Table V-3). That is, values of K_m/D were generated from arbitrarily selected values of d_s/D. The design curves in this appendix are each valid for only one particular value of d_m/D (K_m/D in Chapter V), so that Table V-3 would have to be developed for these specific values of d_m/D rather than arbitrary values of d_s/D. This is demonstrated in Table B-1 below, which replaces Table V-3 in Example V-1 when the design curves in this Appendix are used.

Table B-1*

(1) d_m/D	(2) d_s/D**	(3) d_s ft	(4) d_m ft	(5) d_s/dm
.0945	0.85	3.4	.378	9.0
.2050	0.30	1.2	.820	1.5
.2640	0.10	0.40	1.056	0.4

* Using d_t/D = .25 (see Example V-1)

** Column (2) from Figures B-1, B-2 and B-3.

The number of values in Table B-1 can be increased by using Figures B-4 and B-5. This requires that the 48" circular conduit be converted to an equivalent rectangular conduit since Figures B-4 and B-5 are for rectangular conduits. Note this process can be also used to convert a rectangular conduit to an equivalent circular conduit allowing Figures B-1, B-2 and B-3 to be used for rectangular conduits. The following equation (B-1) is used for determining the dimensions of an equivalent conduit.

$$\frac{Q/W_0H_0^{3/2}}{Q/D^{2.5}} = \frac{1.1}{(y_0/H_0)^{1/3}} \qquad \text{(B-1)}$$

Thus, in Example V-1 the equivalent rectangular conduit would be:

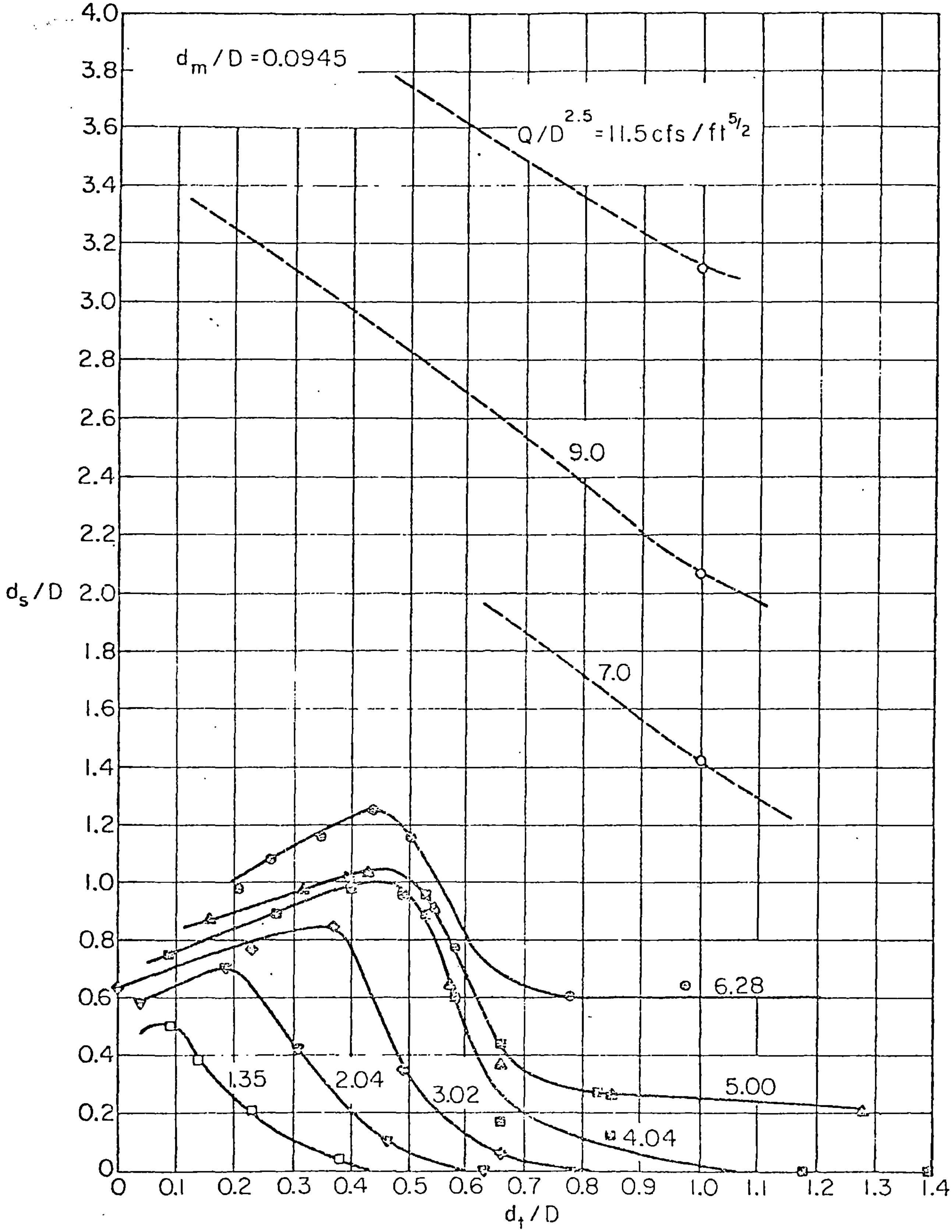
$$\frac{Q}{W_0 H_0^{3/2}} = (3.4) \left[\frac{1.1}{(.63)^{1/3}} \right] = 4.36$$

$$H_0 = D = 4', \text{ so that } W_0 = \frac{110}{(4)^{3/2}(4.36)} = 3.15'$$

From Figure B-4 for $d_m/H_0 = d_s/D = .049$

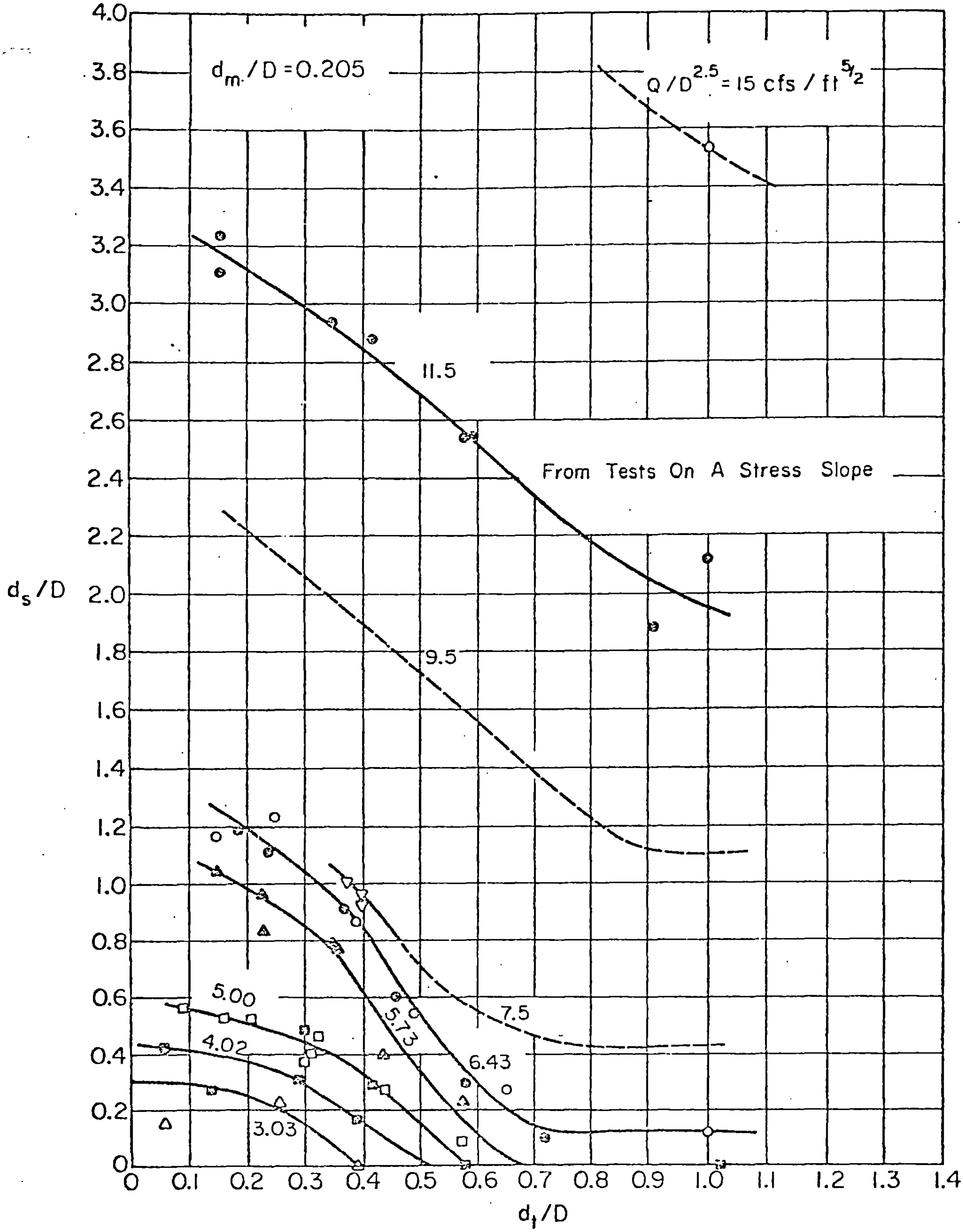
and d_t/H_0 of .25, yields $d_s/H_0 = d_s/D \cong 2.4$.

The width of the scour hole is obtained from Figure B-6 and the length of the scour hole from Figure B-7. Note that the length of the scour hole must be adjusted for the slope of the basin.



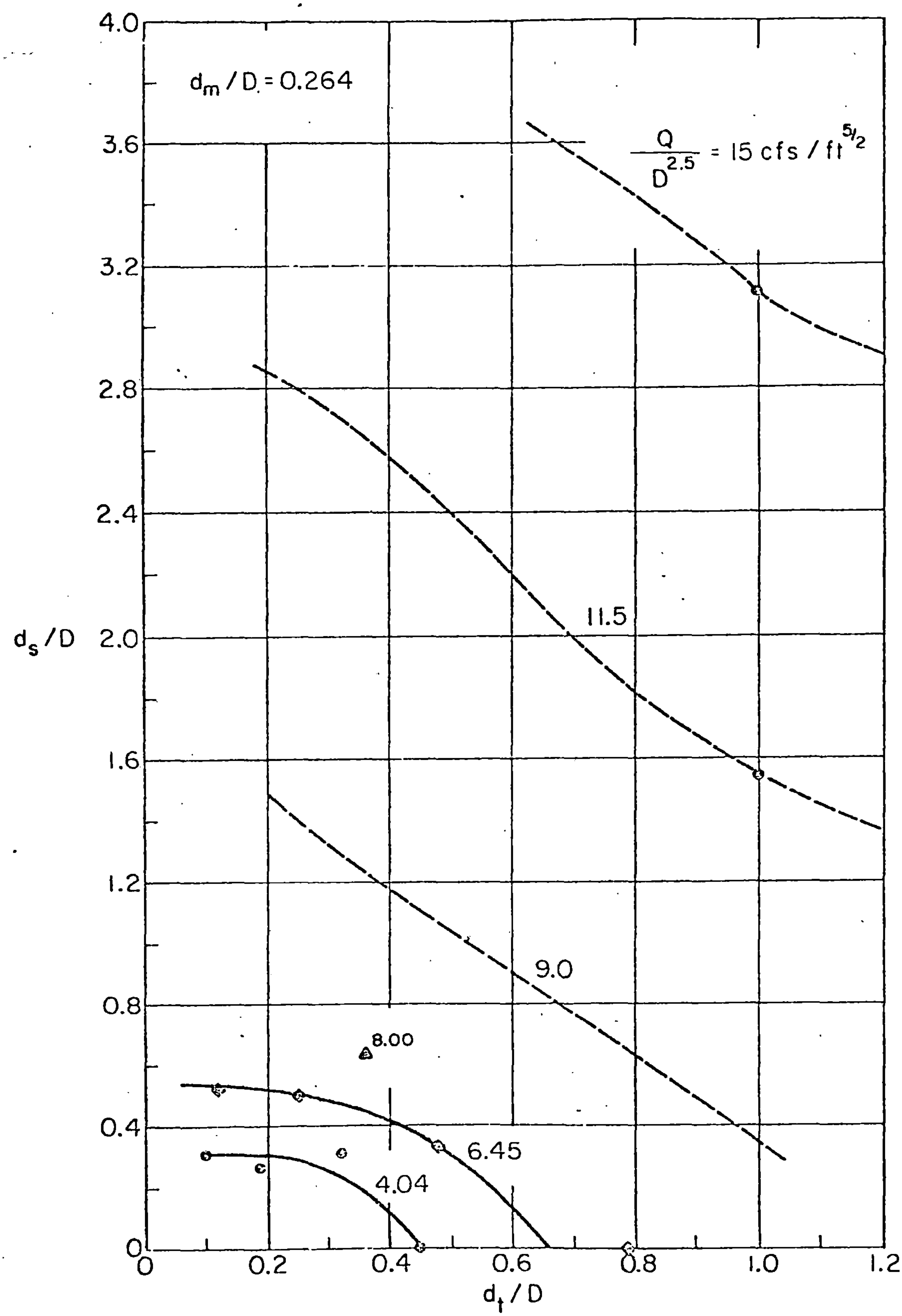
Scour: Plain Circular Outlet

Fig. B-1

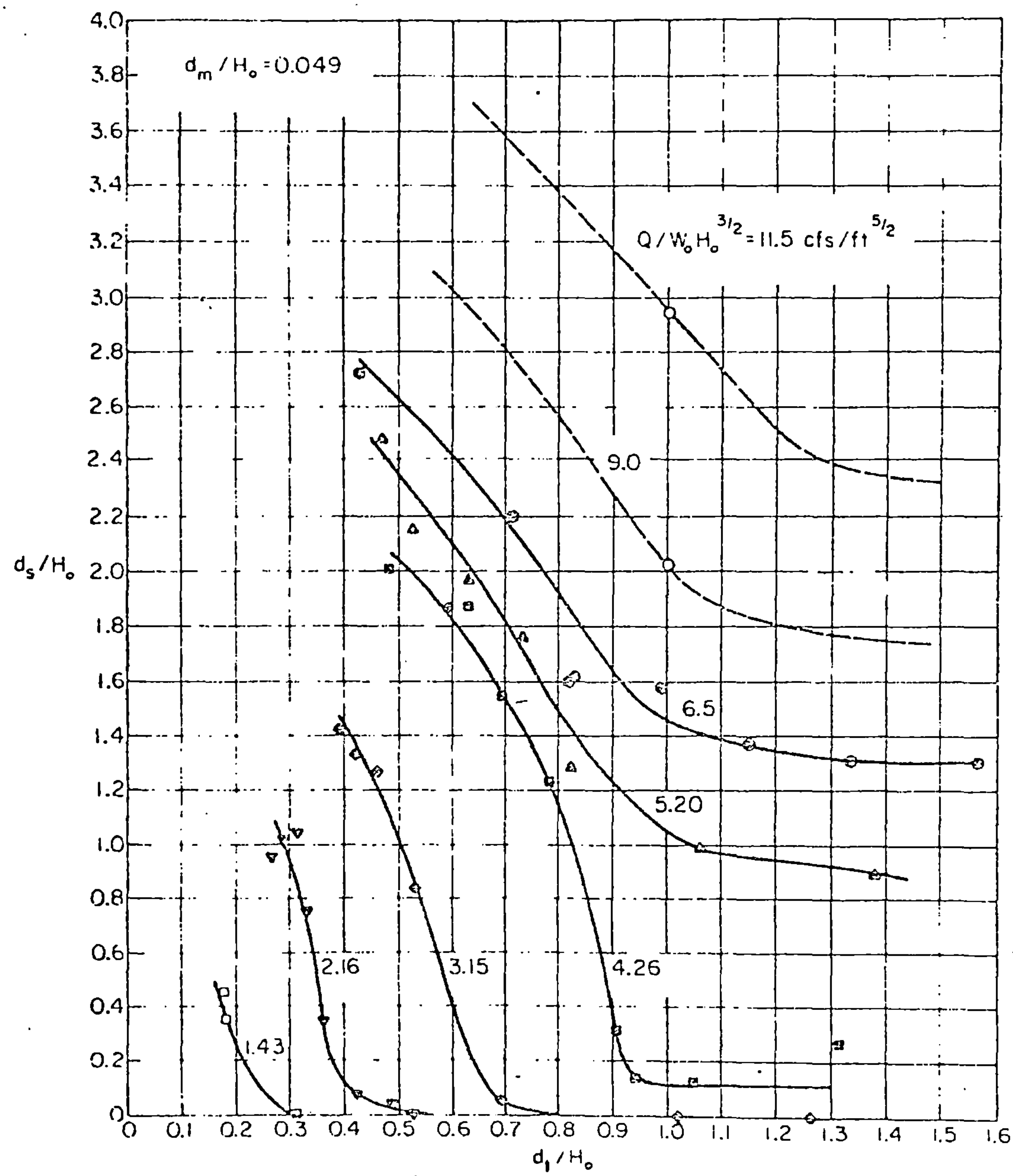


Scour: Plain Circular Outlet

Fig. B-2

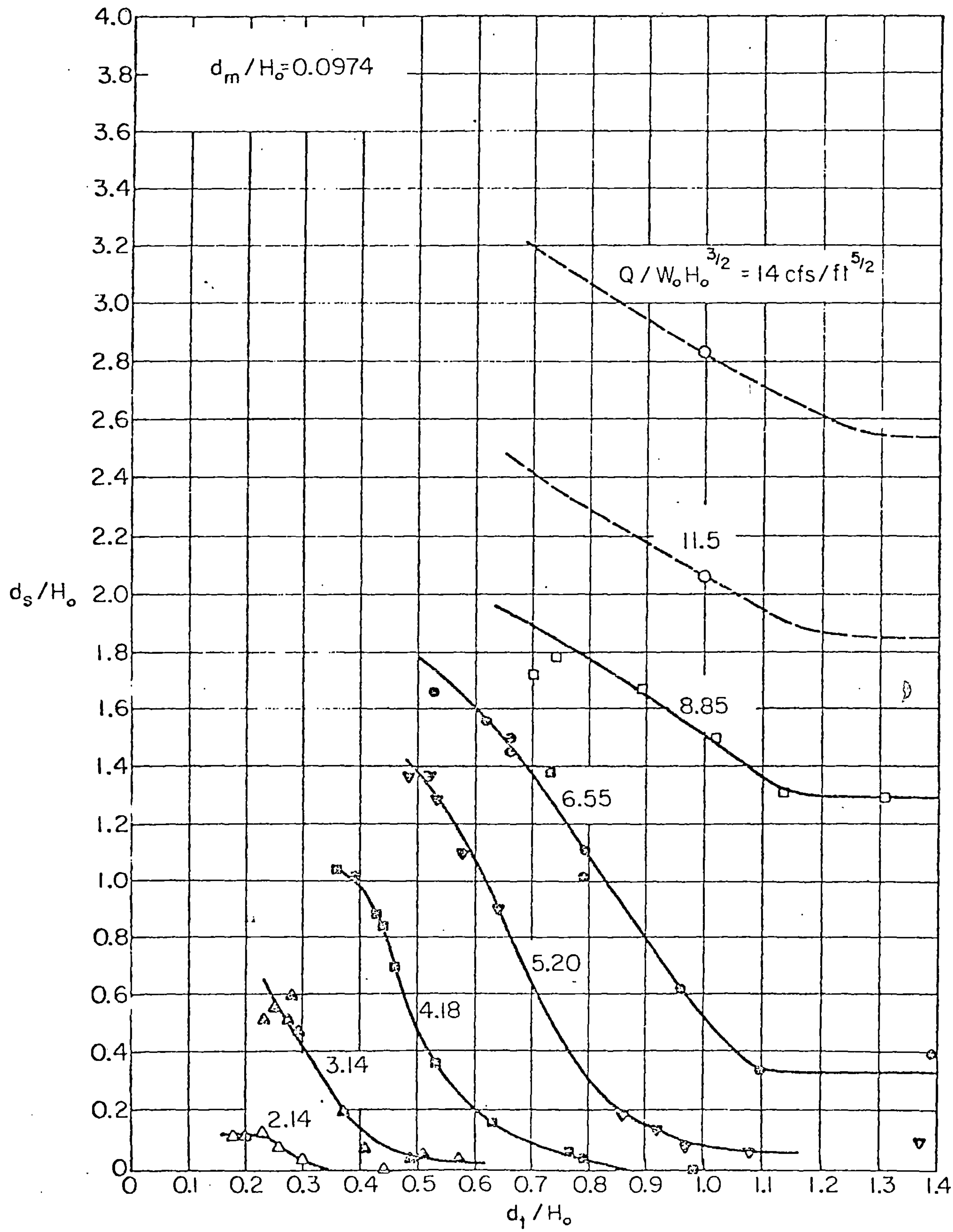


Scour: Plain Circular Outlet
Fig. B-3



Scour: Plain Rectangular Outlet

Fig. B-4



Scour: Plain Rectangular Outlet
Fig. B-5

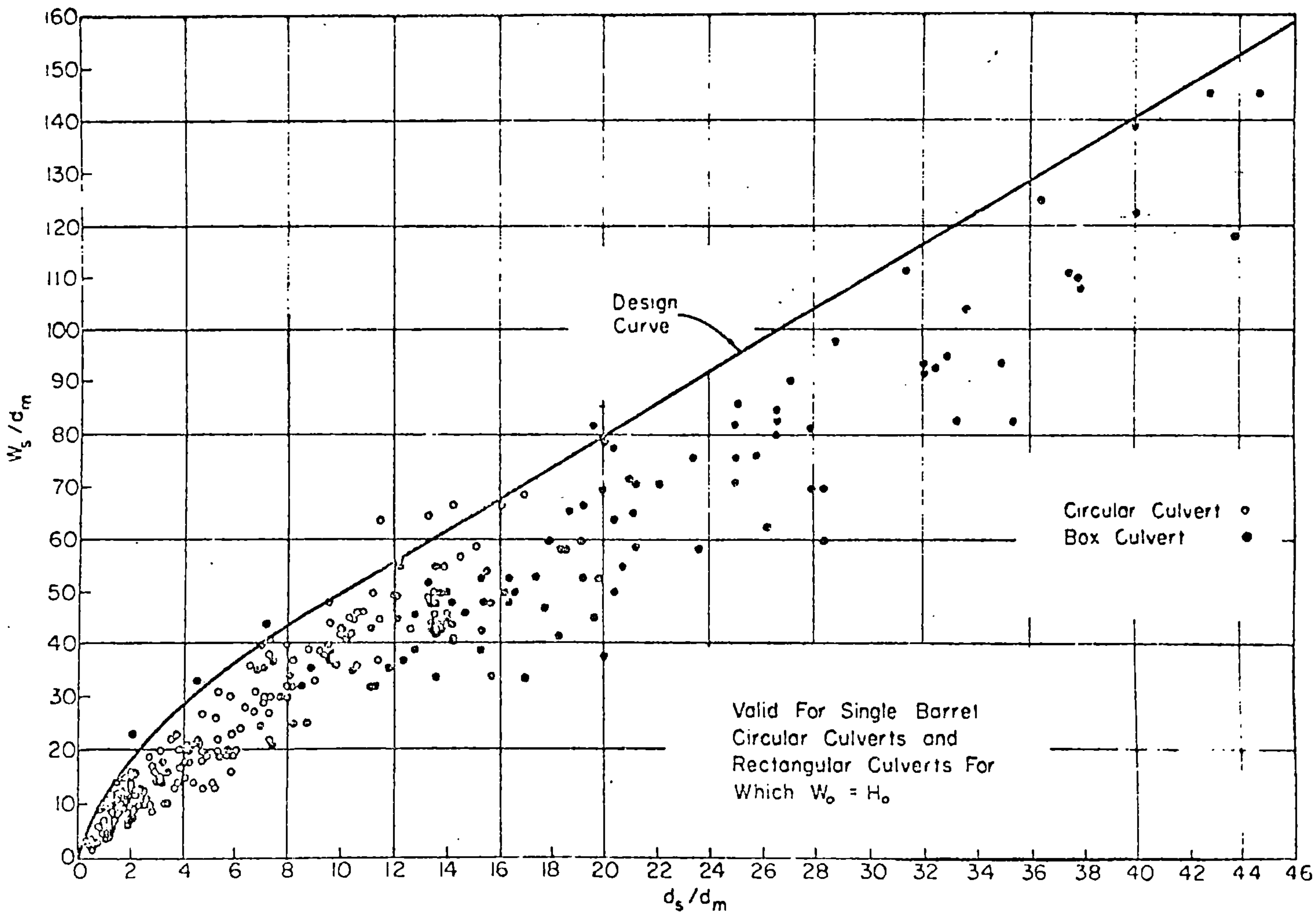
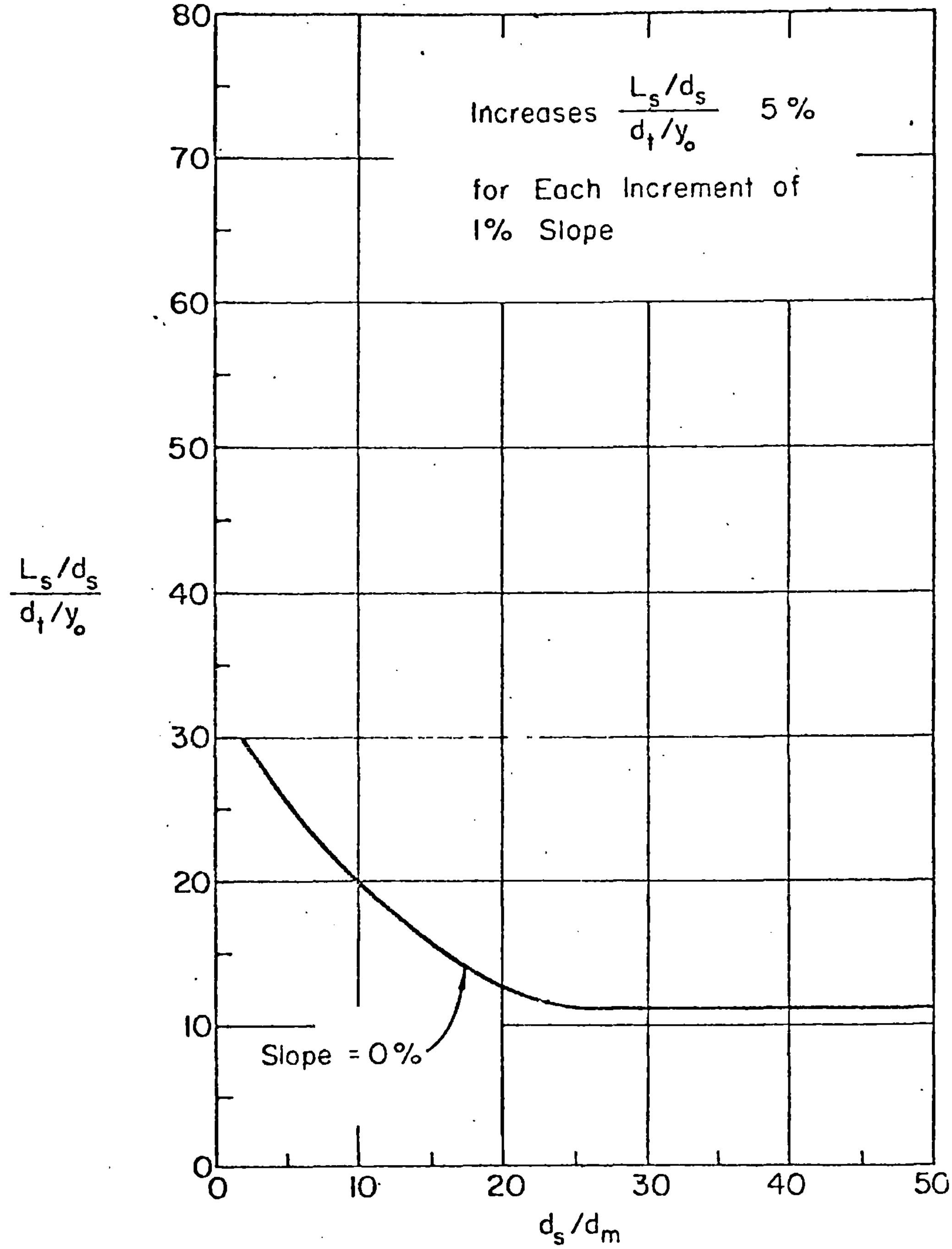
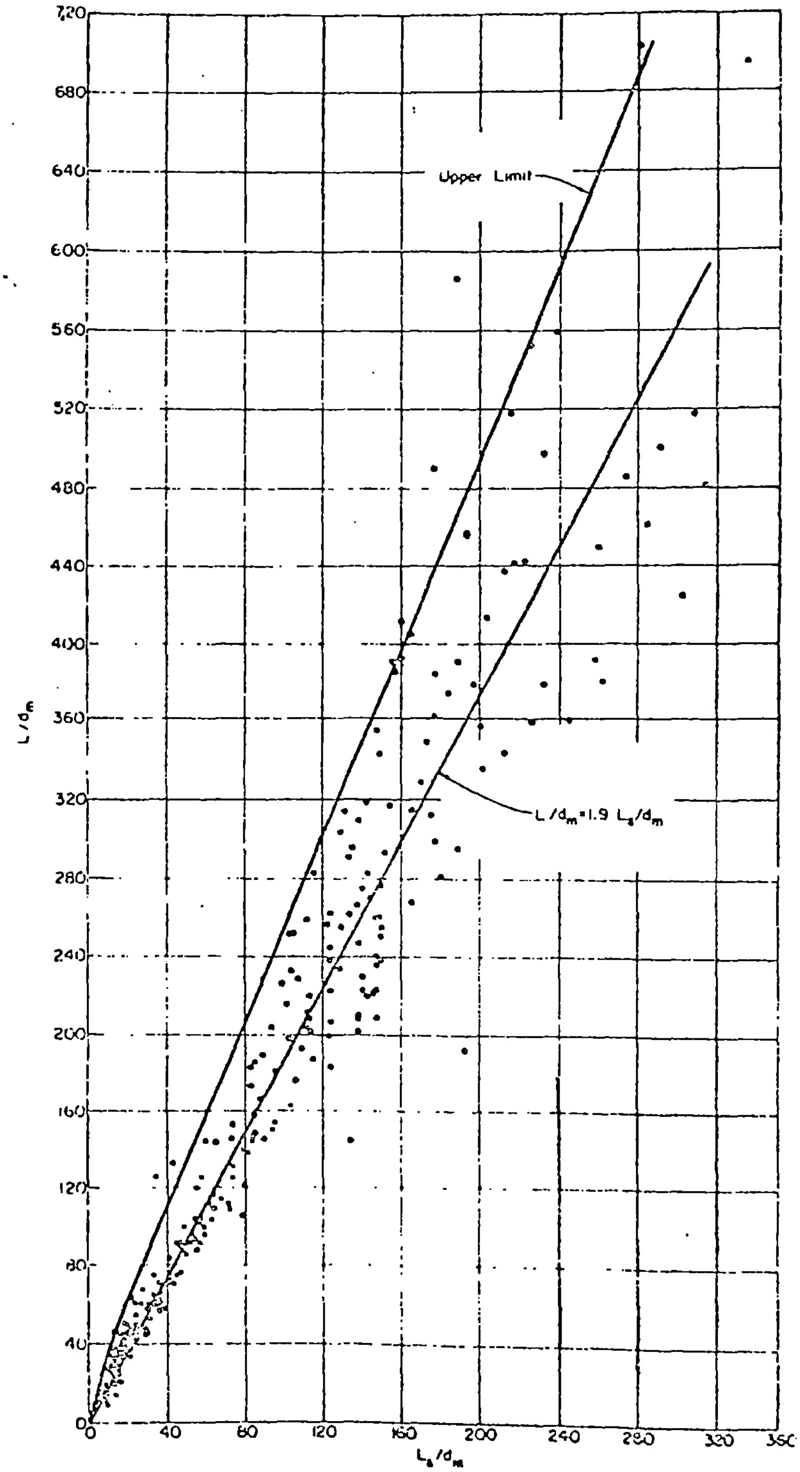


Fig. B-6

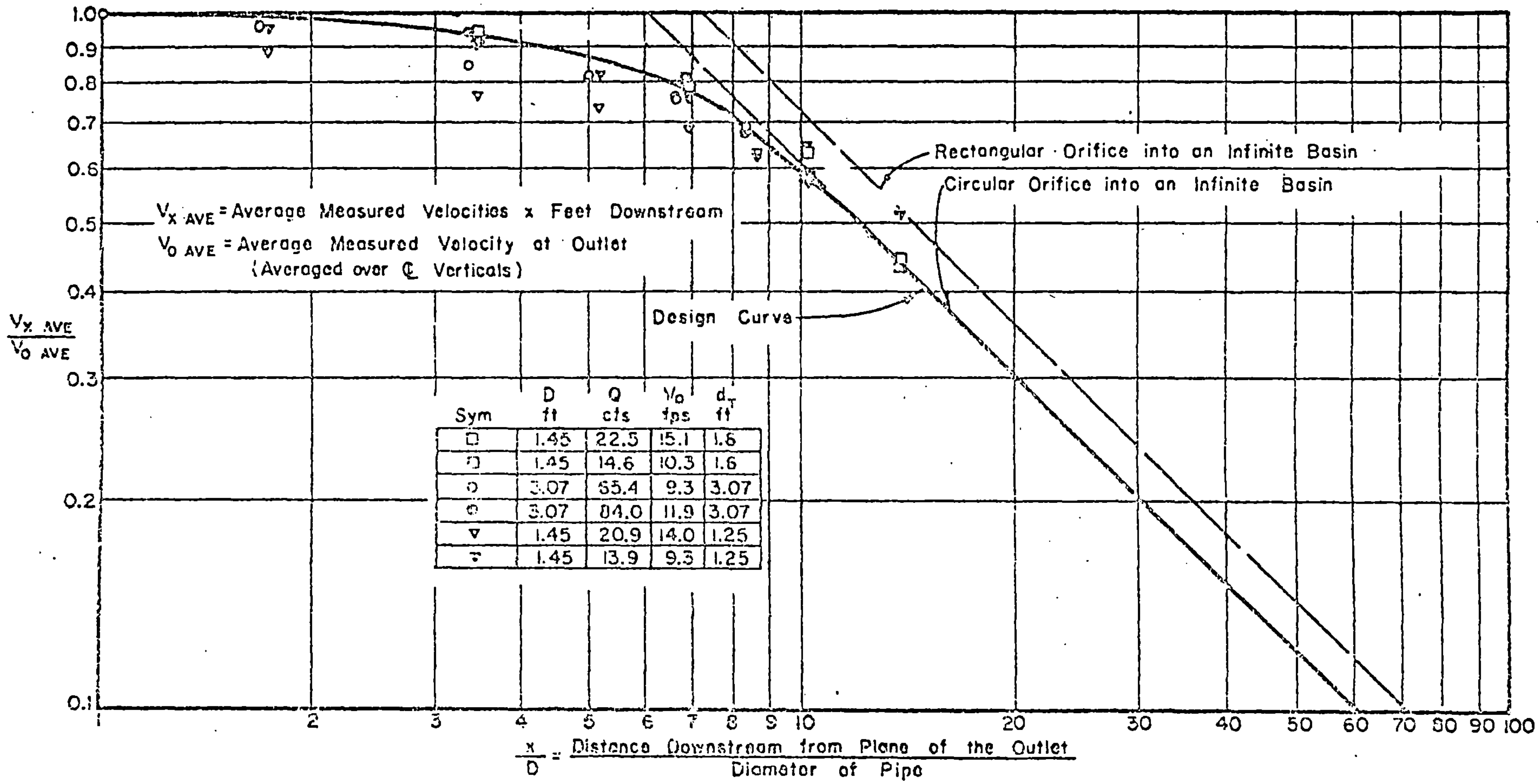
Width of Scour Hole, Plain Outlets



Length of Scour Hole
Fig. B-7



Length of Basin
Fig. B-8



Sym	D ft	Q cfs	V_o fps	d_T ft
□	1.45	22.5	15.1	1.6
□	1.45	14.6	10.3	1.6
○	3.07	55.4	9.3	3.07
○	3.07	84.0	11.9	3.07
▽	1.45	20.9	14.0	1.25
▽	1.45	13.9	9.3	1.25

Fig. B-9

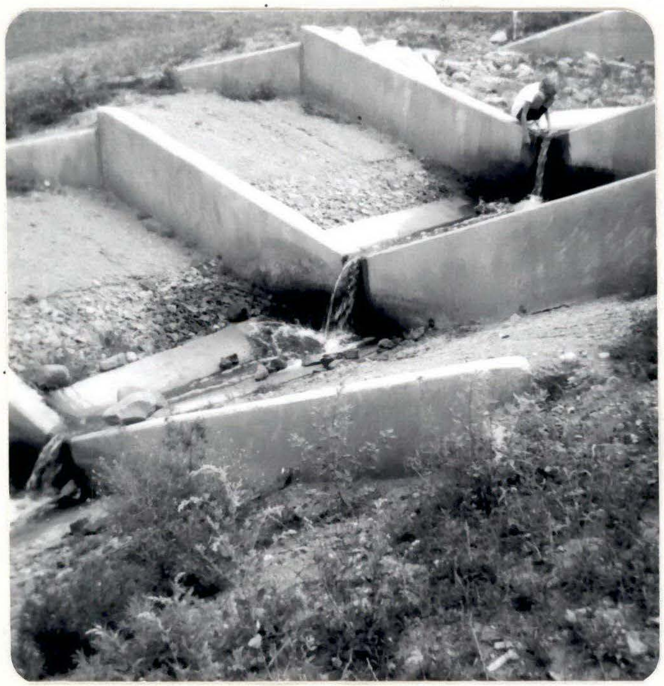
Distribution of Centerline Velocity for Flow from Submerged Outlets

APPENDIX C

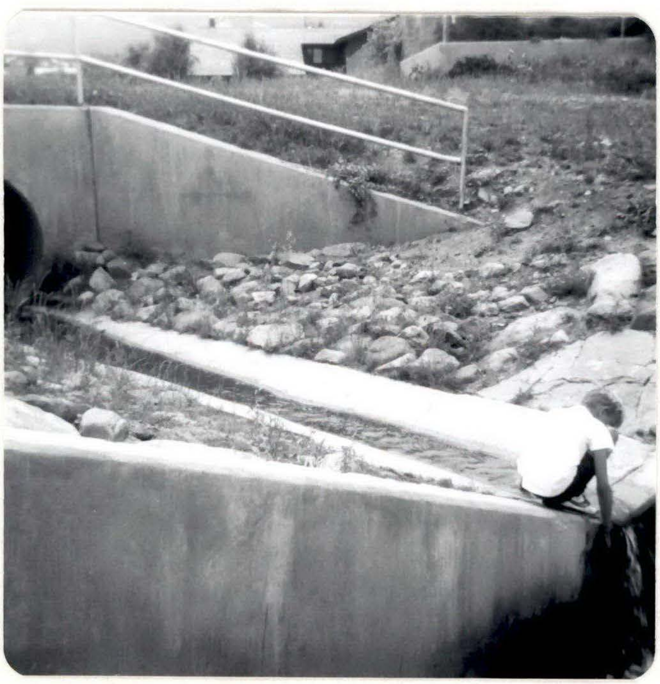
PHOTOGRAPHS

Gabion drop structure with concrete wing walls and basin. Note six inch rock which has been carried into the basin.

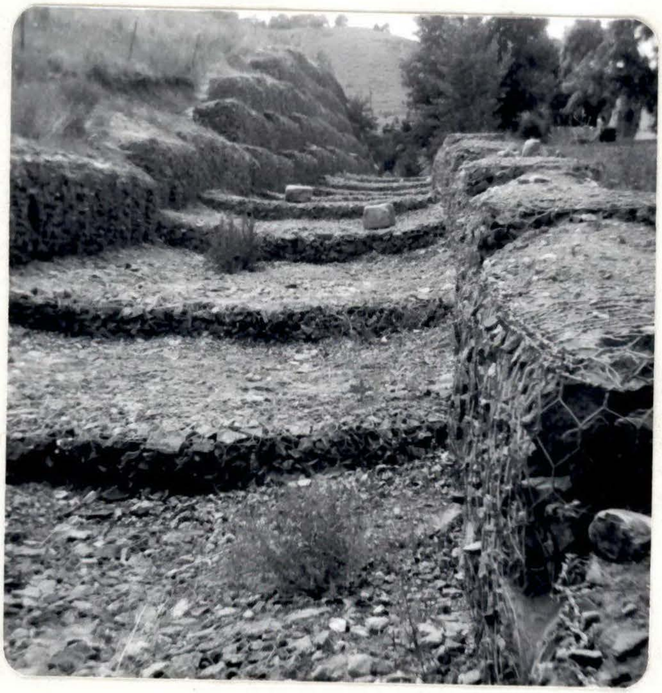
Close up view of the gabion drop structure. Note several strands of wire have been broken by the hammer and anvil action of rocks rolling over the gabion.



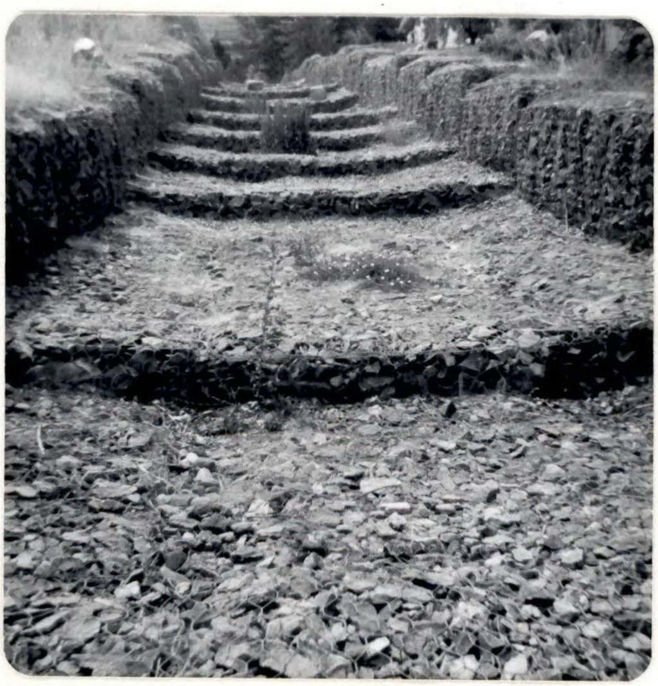
Concrete drop structures with revetment mattress channel lining. Note concrete trickle channel for low flows.



Riprap channel lining above drop structures which serves as erosion protection at the conduit outlet.



Wire encased rock channel lining. The gradient of the rather steep channel is maintained by a series of small drops formed with revetment mattresses.



Note the near vertical side slopes formed with gabions. Also note the large (18 inch) rocks carried by the channel during high flows.

Culvert outlet protected by riprap basin.

Wire encased rock gabions serve as basin walls.

Channel lining formed with gabions (sides) and revetment mattresses(bottom).

Drop structure formed with a gabion. The wire along the upper edge of the gabion has rusted badly.

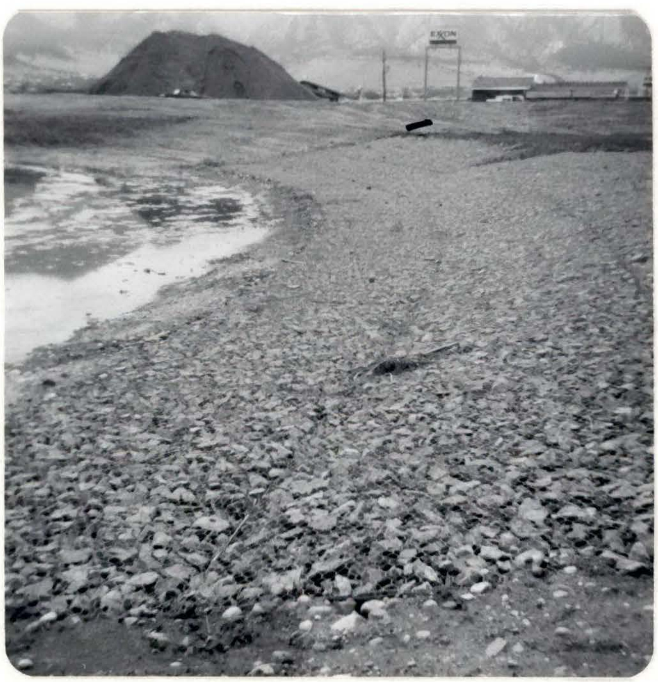
Channel lined with revetment mattresses. Gabion drop structure.

Lining failure resulting from an improper filter is evident in this view.

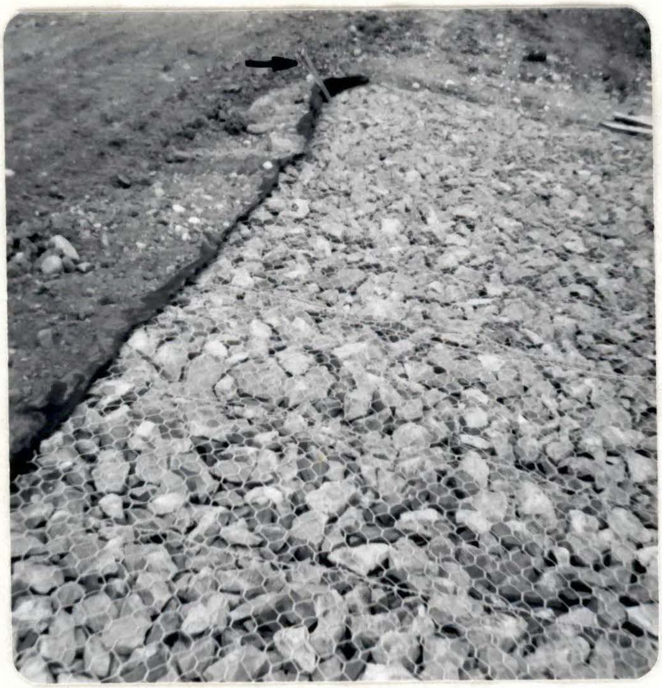
Drop structure and hydraulic jump basin formed with gabions.

Stilling basin with view of downstream sill.

Wire encased rock for erosion protection at a channel bend.



Upstream view of revetment mattress protection at a channel bend.



Filter cloth exposed along the upper edge of a revetment mattress. Note stake at the corner of the revetment mattress.

Riprap check dam.

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